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TRANSACTIONS.

Paper No. 1014.

THE INSPECTION OF TREATMENT FOR THE PROTECTION OF TIMBER BY THE INJECTION OF CREOSOTE OIL.*

By H. R. Stanford, M. Am. Soc. C. E.

With Discussion by Messrs. James C. Haugh, J. L. Campbell, Cliff S. Walker, E. H. Bowser, L. J. Le Conte, John B. Lindsey, Jr., W. K. Hatt and H. R. Stanford.

The principal features incident to the treatment of timber by the creosoting process are the character and quantity of oil which is injected. This paper treats only of the quantity of oil, and is based upon tests and observations, made by the writer, upon yellow pine treated for use in engineering structures in Pensacola Bay.

The depth of penetration and the quantity of injection determine the extent to which creosote oil protects timber against both decay and the attack of sea worms. The cost of the treatment is measured very closely by the quantity of the injection, the disproportionate labor required for low treatments being largely offset by the increased time required by the plant for high treatments. The usual specification stipulates the required quantity of injection in terms of pounds of oil per cubic foot of material.

The depth of the injection appears to be limited by the depth of the sap wood, and the impregnation of the sap wood appears to be

* Presented at the meeting of December 20th, 1905.

uniform, regardless of the quantity of oil which may be injected. Heart wood is practically impervious to oil. Timber having a maximum of sap wood is best suited for treatment by the creosoting process, and uniformity in the character of all timber to be used for any one construction is essential, in order to obtain the uniform treatment of all members necessary for the maximum efficiency and durability of the completed structure. Creosoted material is poorly adapted to resist abrasive forces, as the sap wood is soft and friable, and the heart wood readily splinters or disintegrates in bundles of fibers.

The volume of the material to be treated is the accepted basis for the determination of the quantity of oil which shall be injected. The volume of dimension material can be obtained readily and accurately; the volume of piling is computed after measuring the diameters of the tips and butts, and the lengths, of the various piles, and assumes that the taper from butt to tip is uniform. The actual measurement of each piece of material, and the calculation of the actual volumes, is a rather tedious and monotonous task, and is frequently avoided by the attendant guessing the volume, or, as he would say, he "estimates the volume, from his experience." The guessing method should never be permitted, as the volumes thus obtained are almost certain to be in error, with the result that uniformity of treatment is sacrificed. The assumption that piling tapers uniformly from end to end is not always justified; as an example, the volume of a pile 80 ft. in length, chosen at random, was found to be 59.0 cu. ft. if considered as tapering uniformly, whereas it amounted to 62.7 cu. ft. when calculated from circumferential measurements made at the ends and three intermediate points, showing an excess of nearly 6.3% in the actual volume, as compared with the theoretical volume.

The quantity of water which is mixed with the oil to be injected is a factor which is given very little consideration by inspectors or by the creosoting works. The oil commonly used is about 4% heavier than water at a temperature of 100° fahr., and any water which may be mixed with the oil can only be removed with considerable difficulty. The oil, when delivered at the creosoting works from the refinery, is usually carefully analyzed preliminary to acceptance, but, afterward, the presence of water appears to be of less concern,

and, unfortunately, cannot be detected by either the casual inspection of the mixture before injection or of the treated timber after injection. The writer on two occasions has found as much as 19% of water in oil which it was proposed to use for his work, and in one instance there was 24% of water. It is not believed that water was deliberately mixed with the oil, but its addition was probably gradual and a natural result due to three principal causes, namely: leakage in the steam coils used for keeping the oil liquid while in storage, leakage in the heating coils in the treating cylinders during the injection of oil, and the gathering of moisture from the loads and from the treatment cylinders by the surplus oil which is admitted to the cylinders during treatment and is afterward returned to storage. Dilution is a feature which is not given sufficient care by superintendents, their excuse having been that they had been too busy to consider water.

Heat applied during the treatment appears to be the only feature of the creosoting process which affects the strength of the timber under treatment, and the maximum temperature should be the least which is consistent with the required injection. The process subjects the material to heat during the entire period of treatment, but the maximum temperature occurs during the initial or steaming process. The pressure of the steam applied determines the length of the steaming process, also the degree to which the succeeding vacuum extracts moisture and sap from the load, and has a direct bearing upon the total period of treatment and the capacity of the load to take oil. Results at the Pensacola Navy Yard indicate that yellow pine piling which was subjected to a steam pressure of 40 lb. was very much more brittle and friable than was similar piling treated with the same quantity of oil, but which was steamed at a pressure of 15 lb.

The method which is practiced commercially for determining the quantity of oil injected is very crude and unreliable, and the writer believes that it fails to give even an approximate idea of the injection. Explained briefly, the adopted method predetermines by calculations the volume of oil which a load will require, and reduces the quantity to the equivalent depth, expressed in feet, subdivided to tenths or inches, which would be contained in an elevated storage tank about 20 ft. in diameter. The ordinary cylinder load requires

an injection which will measure approximately, depending upon the size of the load and the unit injection, from 6 to 18 in. in depth in the storage tank. The delivery from the storage tank is measured by an index, controlled by a wire and float, which slides in graduated vertical ways attached either to the outside of the tank or located inside the works. The defects in the method are: lack of refinement, as the result of using a measuring tank of large horizontal capacity; lack of accuracy, due to frictional resistances and elasticity in the movement of the tank gauge, and observed to represent as much as 10% of the total injection; no possible record of losses due to leaking pipes, valves, and cylinder heads, all of which losses deduct from the required injection; elimination of all check on the honest and intelligent manipulation of the complicated pipe system, with its numerous by-passes, during injection.

Contracts for creosoted material are founded upon the weight of oil injected per cubic foot of timber, and the creosoting process should be conducted in such a way that the purchaser may know, with reasonable accuracy, the weight of oil which is given him, and the uniformity with which that weight is distributed in the various pieces of timber. If weight is the basis for contract, then weight is the logical basis for treatment and its inspection. Inspection upon a weight basis requires that three unit weights be obtained: first, the weight of the green timber; second, the weight of the timber after the vacuum process; and, third, the weight of the treated timber. The difference between the first and second weights represents the weight of sap, moisture and volatile substances removed by the steaming and vacuum processes; the difference between the second and third weights is the weight of oil injected. It is impracticable to obtain the weights of entire cylinder loads, and this necessitates the selection of representative members from each load, which may be taken in such number as may be agreed upon by both parties to the contract; this feature involves no material inaccuracy, as all pieces in any one load should be similar in character and of practically the same sectional dimensions, to insure uniform injection. To obtain the weights after the vacuum process would require that the cylinders be opened, causing expense for labor, and reducing the productive capacity of the plant; to avoid these objections, the specification should stipulate a percentage of the green weight to be used

to represent the loss in weight of the timber resulting from the steaming and vacuum processes, and thereby eliminate all hardship and permit definite inspection. The loss in weight from steaming and vacuum treatment probably varies from a minimum of 3% for low steam pressure and heart timber, to a maximum of 15% for high steam pressure and light timber; a little experience will enable the engineer to determine very closely the percentage which should be adopted for any particular treatment and timber.

From the results of various practical tests, all of which were quite consistent, the following is cited, to afford a comparison of the quantity of injection, as determined by the foregoing two methods: The load to be treated consisted of yellow pine piles, each 80 ft. in length, with points not less than 7 in. in diameter, and was to receive 20 lb. of creosote oil per cubic foot. The piling timber was of the species known locally as branch pine. It had been cut about one week before treatment, and had air-seasoned for that period. Two piles were chosen, which were believed to be typical of all others in the load; pieces 5 ft. in length were cut from butt and tip of one of the piles and weighed; the green weight of the second, or entire, pile was also obtained; the two pieces and the entire pile were then placed in the load to be treated. The calculated injection for the load was increased 6%, or 1.2 lb. per cu. ft., to allow for the difference between the actual volume of the piles and the theoretic volume obtained upon the assumption that the piles tapered uniformly from butt to tip; it was also increased 0.2 lb. per cu. ft., to provide for the increased content of the treating cylinder when strained by the final pressure of 125 lb. per sq. in. required to inject the oil; and further increased 1.0 lb. per cu. ft., to provide for losses through leaky valves, etc.; all the additions were unusual and special, and made the treatment what would have been considered, commercially, 22.4 lb. per cu. ft. A steam pressure of 40 lb. was maintained for 15 hours, then a vacuum of 25 in. for 8 hours, after which the cylinder was opened and the butt and tip pieces removed and weighed, to determine the loss resulting from the steaming and vacuum processes; the pieces were then replaced in the cylinder, the cylinder was closed, and the injection of oil was begun. The oil pump was worked under a pressure which was gradually increased to 130 lb., and required 9¼ hours to remove from the storage

reservoir the quantity of oil which had been estimated as equivalent to 22.4 lb. per cu. ft. of load. About 3 hours are ordinarily sufficient to make an injection, but more than 9 hours were required in this case, and during the last hour an equivalent of only about ¼ lb. per cu. ft. of load was taken from the storage tank, indicating that the load was saturated. The records for the two pieces and for the entire pile are given in Table 1.

TABLE 1.

	Butt.	Tip.	Pile.
Diameter, large end	15.0 in.	7.3 in.	14.6 in.
Diameter, small end	14.0 "	6.7 "	8.3 "
Length	5.0 ft.	5.0 ft.	80.0 ft.
Volume	5.75 cu. ft.	1.33 cu. ft.	62.7 cu. ft.
Weight, green	301.5 lb.	70.0 lb.	3520.0 lb.
" after vacuum	273.5 "	61.5 "	3132.8 " estimated.
" " injection	337.5 "	81.5 "	3980.0 "
Loss " vacuum	9.3%	12.1%	11.0% estimated.
Weight per cubic foot, green	52.4 lb.	52.6 lb.	56.1 lb.
Weight per cubic foot, after vacuum	47.6 "	46.3 "	50.0 " estimated.
Weight per cubic foot, after injection	58.7 "	61.3 "	63.5 "
Injection per cubic foot	11.1 "	15.0 "	13.5 " estimated.

The method by tank measurements indicated an injection of 22.4 lb. per cu. ft., as compared with 13.5 lb. per cu. ft. determined from weight observations.

The record of the creosoted piling driven for the support of a wharf at the Pensacola Navy Yard is of interest, as illustrating the uncertainty of the protection afforded by the creosoting process against the attack of sea worms. The timber is of the southern yellow pine variety, and was treated at a plant located in West Pascagoula, Miss., under a contract which required an injection of 20 lb. of creosote oil per cu. ft. The piles were driven during February and March, 1902, in about 26 ft. of water, and were cut off about 4 ft. above mean low-tide level. Inspection of the piling made in July, 1905, showed that, out of a total of 198 support and fender piles, five were worm-eaten, two of them being so reduced in section as to require their renewal. One of the defective piles was broken off at the ground line, and the part above the break was cut

into sections of about 4 ft. each for examination. There appeared to be a sector of the pile of about 90°, and extending the entire length of the piece, which had been penetrated by the worms and from which the sap wood was almost entirely removed. The heart wood was eaten away to a degree varying quite uniformly from about 100% at the water line to 25% at the bottom, 26 ft. below. Above the high-water line the pile was perfectly preserved, but the sector in the sap wood above the worm-eaten portion was much lighter in color than the remainder of the section, and did not appear to contain much oil. Diametrically opposite to the large defective sector, and for a length of about 5 ft. at about the middle of the piece, was a second defective sector of about 30° which was badly worm-eaten, and through which the worms had penetrated to the heart. All sap wood, to its entire depth, appeared to be saturated with oil except in the defective sectors. The other four piles seemed to have defective sectors, similar to those in the first pile, one pile to an equal extent; but in the other three the defects were apparently just beginning to develop.

The following specification is proposed, to govern the creosoting of green stock:

1. Treatment.—Timber shall be subjected to preliminary steaming and vacuum treatments, to obtain the removal of water and sap, and to open the pores of the wood, and shall then receive an injection of — lb. of oil per cu. ft. of stock, which shall be forced into the wood under pressure.

2. Steam Pressure.—The maximum steam pressure during the steaming process, as recorded by a steam gauge, shall not exceed 2 lb. for each pound of oil which is to be injected per cubic foot of stock.

3. Oil.—The oil shall be a dead oil of coal-tar, commonly known as creosote oil. Its specific gravity shall be not less than 1.04 at 35° cent. It shall not contain more than 2½% of water. It shall yield not more than 10% by weight when distilled up to 210° cent. Between 210° and 235° cent., the distillate by weight shall not be less than 25 nor more than 30%, and at least 30% by weight shall not distill below 260° cent. A sample of oil for test shall be taken from the side and near the middle of the treating cylinder after the pump has begun the injection of oil. During the analysis of the

oil, the thermometer bulb shall be kept about ½ in. above the surface of the oil in the retort.*

4. Loading.—All pieces treated in any one cylinder load shall be uniform in character and practically uniform in sectional dimensions.

5. Injection.—The quantity of injection in each load which is treated shall be determined from the relative green and treated weights of one or more full-sized pieces contained in the load, and the determined quantity shall be the difference between the treated weight and the green weight after reducing the latter by two-thirds of 1% for each pound of injection required by contract per cubic foot of load; the number of pieces in excess of one shall be subject to the desire of the contractor. The quantity of injection will be satisfactory if it is within 5% of the quantity required by the contract; no deduction will be made in the contract price for shortage in injection within the 5% above allowed, and no additional compensation will be allowed for injection in excess of the contract requirement, regardless of the amount of such excess.

Air-seasoning is preferable to artificial seasoning, but is usually impossible to obtain, because of limited storage space and lack of time. If timber is air-seasoned, a corresponding reduction should be made in the permissible steam pressure and in the percentage of reduction to be allowed in the green weight when determining the quantity of injection. The percentage of water in the oil can readily be kept within the proposed limits by the exercise of reasonable care.

The tests upon which the foregoing opinions are based were necessarily limited in number. The quality and character of yellow pine varies between such wide limits, even in pieces which are apparently the same, and which have been cut from adjoining stumps, that general conclusions can be safely formulated only after much research, and these notes should be given weight accordingly. The increasing necessity for prolonging the useful life of timber, coupled with the growing faith in the creosote process for attaining that end, justifies careful and scientific study to determine the best

*The specification for the oil is taken from an article by E. H. Bowser, M. Am. Soc. C. E., on "The Preservation of Timber with Antiseptics," which appeared in the *Journal* of the Association of Engineering Societies, for April, 1905.

method of treatment. This paper is submitted in the hope that the experience and observations of others may be brought forth, with the result that future specifications may be prepared more intelligently and that the practice of creosoting plants may be modified, to permit of more certain and definite inspection.

DISCUSSION.

Mr. Haugh. JAMES C. HAUGH,* ESQ. (by letter).—The writer submits the following observations on creosoted Southern Pine piles, brace plank, stringers, ties and caps, which have been in use on the New Orleans and North-Eastern Railroad since 1883.

The timber and piles were treated according to the specifications in use at that time by the Pascagoula Works, the requisite being 15 lb. per cu. ft. The oil used was principally "London Oil." The piles were driven in brackish water, into which the teredo does not come.

These observations represent the experience gained in 22 years of maintenance, and deal with what appear to be the causes of decay and failure.

Round Piles and Poles.—The depth of the injection in round piles and poles is practically limited to the sap wood. The impregnation of the sap wood of round sticks from which the inner layer of bark is not thoroughly peeled is often irregular and defective. After being subjected to the weather, this inner layer peels off, leaving whitish surfaces. Chips cut from these places are almost void of any oil. Even when a chip is put in one's mouth hardly a taste of the oil is perceptible. Piles or poles should have this layer of unformed sap wood thoroughly removed before treatment.

When the heads of piles are cut to grade, an application of hot creosote oil and also asphalt thinned with oil should be applied to the cut surface. The failures of piles on structures with which the writer is familiar can be attributed to neglect to protect the heads of the piles in this way, and to the use of a 1-in. square, bearded drift-bolt. The heart wood decays and leaves the sap wood sound.

Timbers.—Observations on timbers treated for the Lake Pontchartrain Trestle, in 1882 and 1883, show that several varieties of pine timber are still sound and in perfect preservation. No brace plank, whether sap surface or all heart from the center of the log, has decayed, and similar plank laid on the ground for 20 years is sound. In some instances the heart pieces showed but slight penetration. The stringers on this trestle were 6 by 16 in. by 30 ft., and three stringers were packed under each rail. About 6 300 pieces were used, and none has shown any decay, although the quality of the pine varied from the coarsest loblolly, with one sap surface, to the closest and best quality of long-leafed close-grained yellow pine, practically free from sap.

The penetration of oil in these different qualities of pine varied greatly. The loblolly and other coarse-grained stringers show that

* Resident Engineer, New Orleans and North-Eastern Railroad.

timber of this quality absorbed a large percentage of oil, and the close-grained yellow pine a much smaller percentage, probably more than 20 lb. per cu. ft. for the coarse-grained, and less than 10 lb. for the close-grained, pine. The caps used were 12 by 14 in. by 14 ft.; the ties, 6 by 8 in. by 9 ft. The guard-rails were 5 by 8 in. and all were of the same varied qualities of pine. Timbers of the same size were treated together. Mr. Haugh.

There were more failures in caps than in timbers of any other size. This, in the writer's opinion, was due to the size of the timbers not admitting of thorough seasoning, consequently, they "checked" afterward. This checking extended beyond the point to which the creosoted oil penetrated.

TABLE 2.—REPORT OF TRANSVERSE TEST OF CREOSOTED PINE STRINGER, NO. 1, FROM THE NEW ORLEANS AND NORTH-EASTERN RAILROAD.

Breadth = 6 in. Height = 15½ in. at center. Length between supports = 132 in. Maximum fiber distance = 7.75 in. Moment of inertia = 1 861.9. Load applied in increments of 3 000 lb. Riehlé testing machine used. Time, about 1 hr. 30 min. Date, June 23d, 1905.

Load, in pounds.	Stress per square inch in outer fiber.	Deflection. Reading: R.	L.	Total, in inches.	Set. Reading: R.	L.	Total, in inches.	Remarks.
.....	.....	0.10	0.13	.....	.....	.....	.....	11 hard rings to the inch.
3 000	.....	0.16	0.16	0.045	.....	.....	.....	
6 000	.....	0.20	0.21	0.09	.....	.....	.....	
9 000	.....	0.25	0.26	0.14	.....	.....	.....	
12 000	.....	0.30	0.31	0.19	.....	.....	.....	
15 000	.....	0.36	0.36	0.245	.....	.....	.....	
18 000	2 472	0.41	0.40	0.29	.....	.....	.....	Elastic limit.
21 000	.....	0.49	0.48	0.37	.....	.....	.....	For 14 ft. distance between supports, load at elastic limit would be 14 130 lb.
24 000	.....	0.56	0.55	0.44	.....	.....	.....	
27 000	.....	0.65	0.62	0.52	.....	.....	.....	
30 000	.....	0.79	0.74	0.65	.....	.....	.....	
33 000	.....	0.94	0.88	0.845	.....	.....	.....	Maximum, 28 500 lb.

	Actual load, in pounds	Deflection, in inches.	Stress per square inch in outer fiber.
Elastic limit.....	18 000	0.29	2 472
Maximum.....	36 300	0.845	4 990

Modulus of elasticity = 1 597 000 lb. per sq. in., at 18 000 lb. load.

Mr. Haugh. Almost without exception, the caps had the heart core at or near the center, and the circular grains were not cut across in being sawn, as was the case with stringers, plank, and halved and quartered ties.

The stringers were halved from a log in sawing, and all the grains were cut across on the heart face. This permitted more thorough seasoning and the penetration of the oil into the heart face.

The ties were what are known as pole, halved and quartered ties. The pole ties having the heart core in the center also failed by checking beyond the point to which the oil had penetrated.

TABLE 3.—REPORT OF TRANSVERSE TEST OF CREOSOTED PINE STRINGER, NO. 2, FROM THE NEW ORLEANS AND NORTH-EASTERN RAILROAD.

Breadth = 6 in. Height = 15½ in. at center. Length between supports = 132 in. Maximum fiber distance = 7.75 in. Moment of inertia = 1 861.9. Load applied in increments of 3 000 lb. Riehlé testing machine used. Time, about 1 hr. 30 min. Date, June 23d, 1905.

Load, in pounds.	Stress per square inch in outer fiber.	Deflection. Reading: R.	Deflection. Reading: L.	Deflection. Total, in inches.	Set. Reading: R.	Set. Reading: L.	Set. Total, in inches.	Remarks.
.....	.....	0.10	0.15	.....	0.10	0.15	.....	6 rings to the inch.
3 000	.....	0.14	0.20	0.045	0.10	0.15	0	
6 000	.....	0.19	0.24	0.09	0.10	0.15	0	
9 000	.....	0.24	0.29	0.14	0.10	0.15	0	
12 000	.....	0.28	0.33	0.18	0.10	0.15	0	
15 000	.....	0.33	0.38	0.23	0.10	0.15	0	
18 000	.....	0.37	0.42	0.27	0.10	0.15	0	
21 000	.....	0.42	0.46	0.315	0.10	0.15	0	
24 000	.....	0.45	0.53	0.365	0.10	0.15	0	
27 000	3 708	0.51	0.57	0.415	0.10	0.15	0	Elastic limit
30 000	.....	0.56	0.64	0.475	0.10	0.17	0.01	For 14 ft. distance between supports, load at elastic limit would be 21 200 lb.
33 000	.....	0.63	0.74	0.56	0.12	0.17	0.02	
36 000	.....	0.70	0.83	0.64	0.14	0.20	0.045	
39 000	.....	0.75	0.90	0.70	.....	.....	.....	
42 000	.....	0.82	1.00	0.785	.....	.....	.....	
45 000	.....	0.90	1.14	0.895	.....	.....	.....	Maximum, 38 900 lb.
48 000	.....	1.12	1.45	1.16	.....	.....	.....	

	Actual load, in pounds.	Deflection, in inches.	Stress per square inch in outer fiber.
Elastic limit	27 000	0.415	3 708
Maximum	49 600	1.16	6 600

Modulus of elasticity = 1 698 000 lb. per sq. in., at 27 000 lb. load.

Mr. Haugh.

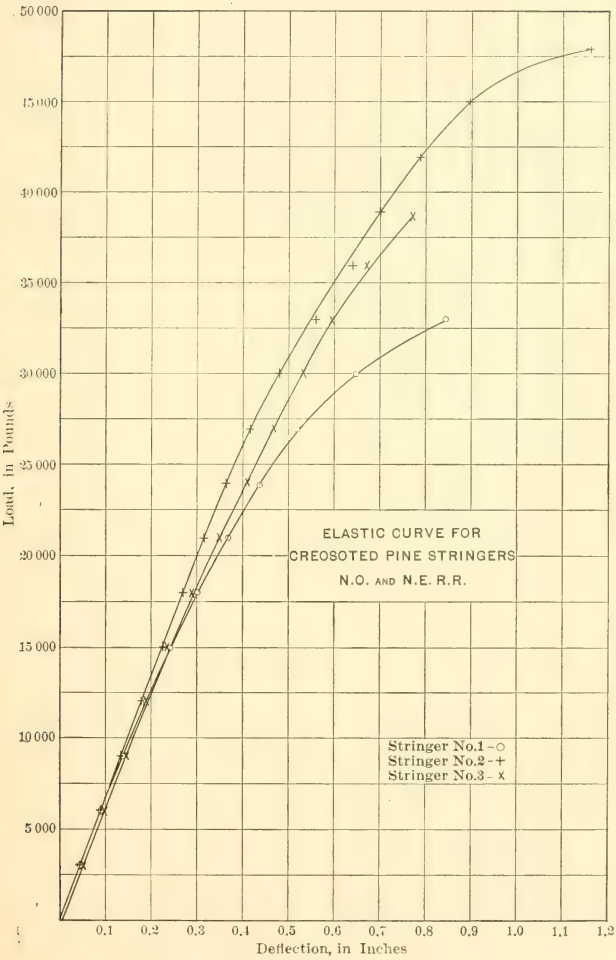

FIG. 1.

Mr. Haugh. The halved and quartered ties having the grain cut across admitted of more thorough seasoning and penetration of the oil, and, when placed in the structure with the "sap side" up, there were few cases of decay.

The halved or quartered guard-rails showed the same durability as the ties. The pieces with the heart core in the center failed.

It would seem from these observations that similar timbers, in which the heart core is in the center, cannot be so thoroughly seasoned as to prevent checking and internal decay, even when the creosoted sap surfaces protect it, and that timbers halved and quartered permit of the proper seasoning, and, even where the quantity of oil absorbed is slight, are still sound.

TABLE 4.—REPORT OF TRANSVERSE TEST OF CREOSOTED PINE STRINGER, NO. 3, FROM THE NEW ORLEANS AND NORTH-EASTERN RAILROAD.

Breadth = 6 in. Height = 16 in. at center. Length between supports = 132 in. Maximum fiber distance = 8 in. Moment of inertia = 2 048. Load applied in incfements of 3 000 lb. Riehlé testing machine used. Time, about 1 hr. 30 min. Date, June 24th, 1905.

Load, in pounds.	Stress per square inch in outer fiber.	Deflection. Reading: R.	L.	Total, in inches.	Set. Reading: R.	L.	Total, in inches.	Remarks.
.....	.....	0.05	0.08	.....	0.05	0.08	.....	5 hard rings to the inch.
3 000	.....	0.10	0.13	0.05	0.05	0.08	0	
6 000	.....	0.15	0.17	0.095	0.05	0.08	0	
9 000	.....	0.21	0.21	0.145	0.05	0.08	0	
12 000	.....	0.25	0.26	0.19	0.05	0.08	0	
15 000	.....	0.30	0.31	0.24	0.05	0.08	0	
18 000	.....	0.36	0.37	0.30	0.05	0.08	0	
21 000	2 700	0.42	0.41	0.35	0.05	0.08	0	Elastic limit.
24 000	.....	0.48	0.47	0.41	0.05	0.09	0.005	For 14 ft. distance between supports, load at elastic limit would be 16 590 lb.
27 000	.....	0.55	0.52	0.47	0.08	0.10	0.025	
30 000	.....	0.61	0.59	0.535	0.08	0.10	0.025	
33 000	.....	0.67	0.65	0.595	0.09	0.11	0.035	Maximum, 32 100 lb.
36 000	.....	0.75	0.72	0.67	.....	.....	.....	
39 000	.....	0.85	0.82	0.77	.....	.....	.....	

	Actual load, in pounds.	Deflection, in inches.	Stress per square inch in outer fiber.
Elastic limit.....	21 000	0.35	2 700
Maximum.....	40 900	0.77	5 260

Modulus of elasticity = 1 405 000 lb. per sq. in., at 21 000 lb. load.

Possibly, boring a hole longitudinally through the center of timbers containing the heart core would allow better seasoning, prevent to some extent the checking of the surfaces, and admit of the penetration of the oil to the inner rings cut. This boring is now done, at a number of saw-mills, in timbers 20 ft. long. Mr. Haugh.

Where the sizes of timbers required are such that proper seasoning is difficult, if not impracticable, built-up members, which admit of this, should be used.

The cubic contents of the charge to be treated and the measurement of the quantity of oil injected should be ascertained as closely as practicable, but, in the case of round pine piles, the sap wood varies from, say, 1 in. thick at the butt, in some piles, to 3 in. thick in others, so that the quantity of oil each pile in a load will take varies greatly, and, if thoroughly seasoned and protected, both are equally durable. The same remarks apply to square pine timbers and in these the absorption also varies greatly. The presence of water and the dilution of the fluid should receive close attention.

Mr. Stanford's remarks as to the inspection of certain piles, in July, 1905—five piles being eaten by worms—leads the writer to attribute the failure of the oil to penetrate certain sections to improper "peeling" of the piles.

The proposed specifications, and the remarks as to air-seasoning, seem to cover the requirements, and the writer would add that if timber is cut from logs which have been floated in creeks or rivers and have been in the water for weeks or months and then air-seasoned, the most desirable seasoning will be obtained. At several creosoting works, this is practicable.

The tests of three pieces of creosoted pine stringers, which were in use on the Lake Trestle from 1883 until taken out in 1905 for test, are given in Tables 2, 3 and 4, and Fig. 1 is a diagram of these tests. The piece, No. 1, was about the average for close-grained yellow pine, having 22 alternate hard and soft grains per inch. The other two pieces are the coarse-grained pine. The penetration of the oil in No. 1 was slight; Nos. 2 and 3 had taken the oil freely. The tests were made by Professor W. B. Gregory, of Tulane University.

J. L. Campbell, M. Am. Soc. C. E. (by letter).—Mr. Stanford's statement that the extent to which creosote oil preserves timber is measured by the quantity of oil and the depth of penetration, while true to the point of saturation with a given grade of oil, should be qualified by the observation that the quality of the oil has quite as much to do with the results as the quantity and penetration. Mr. Campbell.

In the prevailing grade of oil used in the United States the creosote or preserving element constitutes probably not more than

Mr. Campbell. 30% of the volume, from which it follows that about 70% of the injection is of no benefit, and that a large total injection is necessary to secure the required quantity of real preservative.

If the oil was refined to a point where the preservative and non-preservative elements were in a proportion inverse to that above given, as obtains in Europe, the life of treated timber would be prolonged very materially. The use of a better grade of creosote opens a most promising field for improvement in the treatment of timber.

The author's statement, that heart wood is practically impervious to oil, is not confirmed by the experience of the El Paso and Southwestern Railroad in its creosoted long-leaf yellow pine bridge timbers, in which the percentage of sap is very small. Stringers having no sap whatever show a required and satisfactory penetration of oil. As to the percentage and penetration of the preservative element in the oil, the writer is not able to say, but there is nothing to indicate that it is all retained near the surface. This timber is treated under contract by creosoting works in Louisiana.

The writer believes the prevailing method of measuring the quantity of oil injected by the tank gauge to be the simplest and best. Leaky pipes and valves and inaccurate gauges are not a valid argument against the method, but rather are favorable to it, as pipes and valves can be and should be maintained practically tight to prevent useless waste.

From the author's illustration of the method of measurement by weight, it is quite evident that several very material assumptions have to be made, the possible invalidity of which may explain the inadmissible difference between 22.4 lb. per cu. ft. by tank measure and 13.5 lb. per cu. ft. by estimated weight measure.

Certainly, a system of pipes and valves which will permit the escape and loss of 8.9 lb. of oil out of a total of 22.4 lb. ought to be replaced immediately.

The writer is heartily in favor of air- *vs.* steam- seasoning whenever practicable. He finds that treated ties fail because of disintegration due to brittleness induced by the steam-cooking process of preparation for treatment. Treating plants, however, are generally located in the timber belts, where the normal humidity is great and the air-seasoning process quite slow.

El Paso, Tex., the center of the arid Southwest, with its numerous railway lines (several of them transcontinental), and direct competitive connection with vast, though distant, timber belts, would be an ideal place for the air-seasoning process for timber treatment.

With a grade of oil containing, say, 75% of preservative element, thorough air-seasoning, and proper injection of a sufficient quantity of creosote, sound timber, so treated, should lose very little of its

strength and elasticity, and be good for 45 years under ordinary conditions. Mr. Campbell.

CLIFF S. WALKER,* ESQ. (by letter).—The author has given the subject much thought and close investigation, and in the writer's opinion is on the right line to attain the end to be desired by both producer and consumer of creosoted material. Only by thorough injection can timber be preserved, and to approximate that standard of excellence should be the aim of every manufacturer. Poorly treated material has in the past greatly retarded increased demand for preserved timber, and can but injure the future prospects of a growing industry. Any system that tends to give the manufacturer all the profit to which he is entitled, and at the same time protects the consumer, is to be encouraged. The only possible objection to the author's proposed method is that the natural variation in density of timber might cause this method of ascertaining the price to be so risky that in many cases the margin of safety which must be figured would make the cost prohibitive. Mr. Walker.

To arrive at a fair basis would require long and close scientific investigation, and reports on temperatures, pressures, etc., day and night, should not be left to workmen who lack proper training, or even might be ignorant, lazy or malicious.

The writer differs slightly with the author on some points of his proposed specification, principally with the idea of reducing the cost of production. In practice, the writer has never found any advantage in treating timber of uniform size and section at one time, and favors applying the higher temperatures, regardless of size of timber, with the intention of shortening the period of treatment. Close observation has convinced him that a better impregnation of timber is secured.

The writer also thinks that the arbitrary specification for oil is unreasonable, as good results have been obtained with various oils differing materially in specific gravity and fractional distillation; and, as the demand for treated timber increases, every possible source of supply for heavy oil of coal-tar should be open.

On the whole, though, the writer is so heartily in favor of the author's views and conclusions, that he would be highly gratified could he find the time to experiment thoroughly on the lines suggested, using the plant of the Southern Creosoting Company in that work, and thus view results obtained, not in the laboratory, but in actual practice, under all conditions and with all classes of material.

Such investigation and experimentation by one so familiar with the work and the conditions to which the finished product is to be subjected would prove of exceeding value.

* President, Southern Creosoting Company, Ltd.

Mr. Bowser. E. H. Bowser, M. Am. Soc. C. E. (by letter).—This is a much needed paper, in that it touches upon points relating to the subject of creosoting, which, although they have been discussed for many years, are little nearer solution than they were when this method of treating timber was first established.

From their very nature, some of the details of timber treatment can never be brought to any great degree of refinement, though the methods can be improved.

It is well known that sap wood will receive very much more oil than heart wood; that loblolly and old field pine will receive more than long-leaf yellow pine; that in a long pile the small end will receive a greater proportion of oil than the large end, and that the more natural seasoning the material has had, the more rapidly the oil will enter it, and the greater quantity it will hold.

Even after a careful selection of the timber, the quantity of oil per cubic foot injected into each piece will vary greatly, and only a general average treatment can be given, as it is impossible to determine in advance just what material to select for each charge.

By proper inspection, the results of a treatment can be greatly modified and brought much nearer what they should be than with a haphazard method of treating a load without regard to the size, seasoning, or kind of timber.

In considering the inaccurate methods used at present and the proposed change for determining the quantity of oil injected into timber, all clearly presented in the paper, no very definite results obtained from any long-continued and exhaustive experiments can be given, on account of this part of the subject not having been thoroughly investigated, as yet.

A few experiments which show unexpected results, while they are valuable in that they blaze the lines along which investigation should be made, are not conclusive, and sometimes may even be reversed by further experiments.

The writer will take up some of the causes of the discrepancies in present methods, mentioned in the paper, and a few additional causes, under separate heads.

Calculating the Volume of the Material.—In getting the cubic contents of piles, it is the rule to use the diameters of the large and small ends, measured to the nearest inch. It is not often that the measurement is made to the nearest half inch, and taking intermediate measurements is almost unknown.

The taper of the southern pines is remarkably regular, when proper allowance is made for what is known as "swell butts," and in extra long piles, for the more rapid taper, near the point, on account of the small end having been cut above the branch line.

"Swell butts" are caused by the woodman cutting the tree very

close to the ground, in order to get a sufficient diameter, or a longer pile. Mr. Bowser.

To prevent careless measurements of the large end of poles or piles, the Southern Bell Telephone Company, and some others, specify the circumferential measurement 3 ft. from the end.

As the result of a number of measurements taken at different times, and from piles cut in different localities in Mississippi and Louisiana, it has been found by the writer that the taper is very close to an increase of 1 in. in diameter, from the point toward the butt, for every 10 ft. of length.

While the lack of refinement in measuring round timber is often discussed, the errors resulting from taking the mill measurements for unsized sawn timber, in calculating the contents, is usually neglected—in fact, the writer has seen millions of feet of material of this class treated, and not one stick was calculated by the actual dimensions of the cross-section; and, only when the excess in length was over 1 ft., was it usually taken into account; nor would he recommend an exact measurement unless it was stipulated in the specifications, as the bid of the contractor making the treatment is based on the mill size of the section, and the judgment of the inspector is allowed to govern what would be excessive length.

For most purposes for which rough sawn timber is used, the sticks are cut to lengths and the ends squared at the works before treatment, but such is not always the case.

All mills in the Central South set their gauges to saw from $\frac{1}{8}$ to $\frac{3}{8}$ in. full, and this fullness often amounts to $\frac{1}{2}$ in. and even more. This is done to allow for shrinkage after seasoning, irregularities in the alignment of the carriage track, and for the "running" of the saw. Freshly sawn timber will average at least $\frac{1}{4}$ in. full, which in a 12 by 12-in. stick means 4% more than the actual contents. In thin material, such as plank, the variation will average much more. In length, the pieces are seldom cut less than 3 in. full at the mill, and are often more than 6 in. longer than the rated length. Few inspectors will receive a stick which is the least scant in cross-section or length, and, on this account, the variation between the actual and the calculated contents of a charge of sawn timber always makes the cubic content less than it should be.

In the writer's opinion, piles measured at points and butts by the "give and take" method, to the nearest half inch, give much more accurate results than sawn timber taken at the size for which it was cut.

Allowable Quantity of Water in the Oil.—The specifications for oil, quoted in the paper, are, as far as the water is concerned, specifications by which oil should be bought; and it is the common custom to allow $2\frac{1}{2}$% of water, and no more, in the oil from the manu-

Mr. Bowser. facturer, but, in the writer's judgment, a margin should be allowed for water taken into the oil during the process of treatment at the creosoting works, and at least 5% should be allowed, provided the quantity greater than 2½% is compensated for by an extra injection of oil.

Many specifications allow a maximum of 8%, and this quantity does not seem to be excessive, as the quantity of water would be offset by deeper penetration or by putting the proper quantity of oil in the same space that it would occupy if there were not more than 2½% of water.

With present methods of manipulation, it is not practicable to keep the water to a 2½% limit at all times. Getting water out of oil is expensive, and rigid specifications would, no doubt, cause a corresponding increase in prices if the inspection for water was also rigid.

Waste During Injection, and Loss Due to Reduction of Volume of Oil under Pressure.—Very nearly all the waste oil can be accounted for by collecting the leakage in large pans made for the purpose, and emptying it into the measuring tank at the completion of the treatment. If there is a leak in the steam coils, this is easily discovered, and the set of coils in which it is found can be cut out of service and throttled at the end where the oil would pass out.

The writer has no data at hand showing the elasticity of the oil, and can form no idea as to the reduction of the volume caused by the pressure to which it is subjected, but it probably does not amount to very much.

In some specifications the payment for the quantity of oil injected is based on the difference between the readings of the gauge before the oil is turned on the charge and after it is pumped back into the measuring tank. This eliminates any errors due to compression, the oozing out of the oil from the wood after the pressure is released, and very nearly all the loss of oil by waste, if the waste is properly collected.

Such provisions were made in the specifications written by J. F. Coleman, M. Am. Soc. C. E., for the treatment of material for the New Orleans docks.

The Prevailing Method of Measuring the Quantity of Oil Injected.—A number of the creosoting works have measuring tanks 20 ft. in diameter, but tanks proportioned to the sizes of the cylinders would give more uniform results. It has been the writer's experience that a well-constructed sliding gauge, kept in good condition, can be read within less than ¼ in., and a 6-in. treatment from a 20-ft. tank ought not to vary more than 3% either way.

The heavier the treatment, or the larger the cylinder and load, the less will be the variation.

A tank, 30 ft. deep, with a diameter great enough to give a capacity of one and one-quarter times that of the empty cylinder which it supplies, would be about the right proportion, and, with a properly made sliding gauge, ought to give results within 1% of a refined measurement. Mr. Bowser.

This rule would give, for a cylinder 6 ft. in diameter and 100 ft. long, a measuring tank 11 ft. in diameter; and, for a cylinder 9 ft. in diameter and 100 ft. long, a tank 16 ft. in diameter.

The accuracy of the sliding gauge will depend upon the size of the horizontal section of the float, the frictional resistance in the bearings of the pulleys, the pliability of the wire connecting the float with the sliding pointer, and the proper balancing of the pointer so that the guides will not clamp on the gauge-board.

The larger the float the less the distance it will be lifted out of the oil by the friction of the pulleys and the pointer guides.

The pulleys, of course, should be large enough in diameter to prevent the tendency of the wire to form a hook. A light, well-made chain would give better results than a wire.

The wind playing on a long wire running from a measuring tank to a gauge inside the cylinder shed has been observed to make a variation in the reading of the gauge.

The difference between the quantity of oil injected into sawn timber, as indicated by present methods of measurements, may often vary as much as 10% less than the actual amount. This is due to the following causes:

The difference between 8.7 lb. per gal., the weight of the oil at about 75°fahr., which is generally used in calculating the quantity of oil to be injected, and 8.33 lb. per gal., the weight of the oil at 180° fahr., which is about the temperature of the oil in the measuring tank, making a shortage of oil amounting to 4%; the difference in the volume of the oil when in the measuring tank and when under pressure in the cylinder; the running out of some of the oil from the timber after the pressure is released; the fullness of the timber not being taken into account; and the loss by waste, which can be kept very low if proper care is used. There can be considerable loss if care is not taken in analyzing for water.

With proper specifications and proper inspection, all the foregoing discrepancies can be reduced to a very small percentage.

Proposed Method, of Estimating the Quantity of Oil Injected, by Full-Sized Test Pieces.—The most radical departure brought forward in Mr. Stanford's paper is the proposed change in the method of determining the quantity of oil injected.

The quantity of oil which different pieces of timber will absorb varies greatly with the texture, the quantity of resin in the ducts, the quantity of sap wood, the seasoning, the relation between thickness and breadth, and the length.

Mr. Bowser. It is so well known that sap wood will absorb very much more oil than heart wood that nothing further need be said. The variation in the quantity of oil that can be absorbed on account of different degrees of seasoning is very great.

In a 16-lb. treatment, at one works, a load of branch pine piles, 50 ft. long, which had been seasoning for 6 or 8 months, was given a treatment in about one-third the time usually taken for freshly cut timber. The inspector allowed three piles, only a few days from the woods, to be put in with the seasoned piles, and, after treatment it was proved by boring that the latter piles were well treated to the center and the former were penetrated by the oil about 1 in. only, and that that space was not very well saturated. This, of course, is an extreme case.

The writer has often observed that the oil can be injected into piles which have had only a week or two of seasoning much more rapidly than when the piles are put into the cylinder fresh from the stump, or are taken out of water storage.

Piles allowed to lie in the sun, on ground more or less wet, will not only show a different degree of seasoning on the upper and lower sides, but, after creosoting, will show very plainly a difference of saturation. This, no doubt, was the cause of the irregular treatment of the piles eaten by marine worms at the Pensacola Navy Yard. These piles were from West Pascagoula, and nearly all were delivered at the works by water. In hauling them out, for sorting an order, while most of them were put on skids, some were allowed to lie on the wet ground without occasional turning.

The relation between the cross-section and the cubic contents of timber gives large variation in the absorption. The absorptive power of a 12 by 12-in. stick, compared with a 1-in. plank, when the treatment would give about $\frac{1}{2}$ in. of penetration, would be as 16 to 100.

In sticks of the same cross-section, the shorter ones will absorb more, on account of the penetrating power of the oil being greater in the ends of the fibers. In heavy treatments the oil will penetrate as much as 1 ft. into the end of heart wood if it is not very resinous, and it will sometimes penetrate 5 or 6 ft. into the ends of sticks of loblolly and old field pine.

No matter how carefully a charge is inspected, it is only possible to get an average treatment. The writer doubts that human ability can select from a promiscuous pile of round or square timber a cylinder load from which two or three or all the pieces could be weighed before and after treatment so as to give a basis of measurement for oil which would not often vary as much as 100% or more, from the actual quantity in the timber. Future investigation may show whether or not this doubt is correct.

Here is a case in point, where 80-ft. piles were gauged by 5-ft. test pieces: In the calculations for the quantity of oil to be injected into the piles, 12% additional was allowed for discrepancies in the measuring system. On account of the short pieces having a much greater proportion of end wood exposed than the piles, the result should have shown a greater quantity of oil than by the tank measurement, if the absorbing power of the different piles was approximately equal. The allowance for the inaccuracies of the measuring system would seem to be very close to what it should have been, and, no doubt, very nearly the correct quantity of oil to treat the timber properly passed from the measuring tank into the cylinder, but, according to the test pieces, 37½% of the oil was missing. From the construction of the plant, the oil must have gone either into the wood or into the underground dumping tank through leaking valves, and the only point to be settled is—which? Mr. Bowser.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer's experience in this matter has been confined to operations on the Pacific Coast, where the *modus operandi* is quite different, in many respects; but, nevertheless, so far as he is able to judge, the final results seem to be about the same as for the method reported by the author. Mr. Le Conte.

In 1890, when creosoting works were fairly started in California, the writer was classed among the stalwart advocates of this process of preserving timber. He was untiring in his efforts to reduce the uncertainties of the process to a minimum, and, with this end in view, studied carefully the minutest details of operations from the very beginning to the final taking out of the finished product. After the most thorough research he is free to state that the very best creosoting works in the country, using the very best grade of oil and performing the operations in the most thorough and conscientious manner cannot turn out a uniformly good product. That is to say, the general output of the works, figuratively speaking, may be classified as follows, according to the degrees of imperfection:

First.—One-quarter will be found, on examination, to be first-class in every particular, with no defects.

Second.—One-quarter will be slightly imperfect, but would easily pass inspection.

Third.—One-quarter will barely pass inspection.

Fourth.—One-quarter will not pass inspection at all, and would have to be put into the boiler for a second dose, or sold to some one who is not so particular. All four classes have had the same treatment, administered by the same competent men and at the same time, and yet the final results are widely different. There can be but one explanation for this unavoidable state of affairs, and that is, the natural variations in the physical character of the timber;

Mr. Le Conte. nothing else will account for it. It is extremely difficult to cull out the inferior timber before preservation, and, as a rule, it is never done.

The natural variations can be brought out most graphically by taking a condemned pile, cutting it into 2-ft. lengths, and then critically examining the sections made by the saw. Many years ago, when the writer first looked at them, they threw a hopeless cloud of doubt about the efficiency of the entire process. Long experience, however, has toned down these unavoidable difficulties very materially, so that now if a uniformly good job is demanded the timber must be fresh cut green timber, free from physical defects, and selected with the greatest possible care. Even then, physical defects will crop out in spite of every precaution. In the hurry of everyday business, and especially when the superintendent of the works has a rush order, care in selection of material is simply out of the question; at all events, such care is never taken, disappointment is sure to follow in a few months, and the creosoting plant is "given a black eye," so to speak.

In preparing specifications, there is just one thing to keep constantly in mind, and that is, one cannot, by any set of specifications, obtain a better product than the creosoting plant is physically able to produce. This is the business limit beyond which one cannot advance. Therefore, in preparing specifications, the first thing to find out is what the creosoting plant is physically able to do; then the specifications should be framed so as to compel the works to comply with them.

In California the treatment is quite different from that described by the author, and a brief statement may be of interest.

The timber to be preserved is loaded on heavy iron trucks and run into the boiler, and then the ends are closed tight.

All the upper cocks are then closed and the three bottom cocks are opened and the vacuum pumps started. The hot creosote oil (130° fahr.) rises from the ground-tanks and gradually fills the boiler. The foreman watches the filling by feeling the rise of the hot line on the shell of the boiler until it is nearly full of hot oil. Then he stops the vacuum pumps, closes the three bottom cocks and opens a 2-in. safety cock on top of the boiler. He then completes the filling of the boiler with an auxiliary force pump and watches the filling until complete, by means of the safety cock.

When the boiler is full, superheated steam is turned into the steam coils, in the lower half of the boilers, the temperature of the contents being raised from 130° to 220° fahr., and maintained at that temperature for a period of about 10 hours. The vapors of sap and moisture from the timber are blown off through the safety cock on top of the boiler. As long as the sap vapors are rising and dis-

Mr. Le Conte.

charging the temperature is easily held at 220° fahr., but as soon as these vapors are all driven off, the temperature rises rapidly and the vapors of naphthalene begin to blow off, and, condensing, fall like snowflakes about the boiler room. Vaporization is finished. Auxiliary pumps are started once more and the boiler entirely filled.

All cocks of every description are now closed tight, and, for the first time since operations began, the pressure process begins. The measured quantity of oil, previously calculated, is then forced in with force pumps, the time required depending upon the character of the timber being treated, generally from 4 to 5 hours. The pumps keep up a steady pressure of 150 lb. per sq. in. on the boiler, and the steam coils below maintain a steady temperature of about 200° fahr. When the measured quantity is forced in, the process is completed.

The total time of treatment from beginning to end generally approximates 16 hours. The depth of penetration and quantity of dead oil are the main features. On the Pacific Coast the specifications generally call for 12 to 14 lb. per cu. ft. The writer prefers heavier doses; and, furthermore, that the penetration of the black oil shall not be less than 1 in. in depth. This requirement arises from the fact that the oil, while being forced into the timber by pressure, undergoes a mechanical separation, the lighter and more fluid tar-acids and naphthalene penetrate through the full depth of the sap wood, while the heavier portions, mostly the residuum, remain near the surface. It is the latter which, to his mind, constitutes the main protection against the teredo.

The author refers to the danger of the dilution of oil with from 19 to 24% of water. This danger could hardly arise in the California process, in which the green timber is boiled in oil at 220° fahr. for 10 hours at a stretch, or until all watery vapors disappear. This is the highest temperature to which the timber is subjected at any time. The author's suggestion that the weight of creosote due to impregnation is more reliable than the volumetric tank method now in vogue would hardly be practicable in the California practice.

The author's experience at Pensacola, where only 5 piles out of 198 were badly worm-eaten after 15 months' exposure, seems to the writer to be a very fair record, indeed.

Mr. Lindsey.

JOHN B. LINDSEY, JR., ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Stanford emphasizes the point of view that, contracts for the treatment of timber and piling being based on the weight of oil injected per cubic foot, the logical basis for inspection should be the weight injected per cubic foot. It is extremely doubtful, however, whether any system short of actually weighing the entire cylinder load of timber, both untreated and treated, would be considered favorably either by the management of the treating plants or by those who use

Mr. Lindsey.

creosoted material. The structure of different pieces of timber varies too greatly to permit the adoption of any average sample-piece method. The writer has noticed a marked difference in the treatment of different sections of the same pile, and occasionally a marked difference in the penetration on the same section. In most cases the closer grain of the less penetrated section explained the difference, but in some cases there was no apparent cause to explain the lack of uniform treatment.

The teredo-eaten pile at Pensacola, described by Mr. Stanford, with a defective 90° sector of sap wood, seems to be a practical example of the uncertainty of securing uniform treatment along the entire length of a pile where the percentage of sap wood to heart wood is much more constant at every cross-section than in the case of a sawed stick, and where conditions as to steam pressure, degree of vacuum and oil pressure were identical.

How much greater, then, does the uncertainty of uniform treatment become when the treatment of individual sticks is considered. The stick with the greater proportion of sap to heart wood, and more open grain, may receive a 20% greater injection than another stick in the same cylinder load.

Clause 4 of Mr. Stanford's proposed specifications, to insure the uniform size and structure of the pieces in each cylinder load, although entirely approved by the writer, is a difficult one to carry out fully in actual practice. This difficulty is especially great at present, as there is such an unprecedented demand for all classes of structural timber and lumber.

The percentage of the total weight to be assumed as becoming seasoned during the steaming and vacuum periods would be another vexing problem, difficult to determine equitably. The question as to whether 8% or 15% of the weight of green sticks in a load would be seasoned would mean a difference of about 4 lb. in the treatment. Such a consideration, in a plant not operated conscientiously, would have a tendency to reduce the effectiveness of the steaming and vacuum periods, affecting economy of fuel and securing a greater estimated injection of oil than actually made.

In the future the greater portion of creosoted material needed will consist of cross-ties, telegraph and telephone poles and cross-arms. Such standard-size stock will doubtless be, to a large extent, air-seasoned before treatment. Operating upon air-seasoned stock, the steaming period would be reduced to a brief interval for sterilizing the timber, or perhaps be entirely omitted. The seasoning percentage factor would then be reduced to a minimum, and ticable to weigh each section of the load before and after treatment. with stock, say, not more than 50 ft. long it would be entirely prac- The injection could thus be determined by the estimate of the weight of oil injected in the entire cylinder load.

Mr. Lindsey.

O. Chanute, Past-President, Am. Soc. C. E., in his valuable paper on "The Preservation of Railway Ties in Europe,"* gives the following interesting evidence of the care exercised by the German plants in treating ties:

"The most notable thing in Germany is the painstaking care with which every operation is performed. The ties are not treated until they are thoroughly seasoned, this generally takes six months to one year after cutting and piling in open piles in the yards, most of which yards will hold one year's supply. The chemicals are tested constantly, a laboratory being attached to each plant, each buggy load of 32 ties is weighed before and after treatment, to make sure that the ties have absorbed enough and every little while each individual tie of a buggy load is weighed in and out."

Present Methods.—The methods used to determine the quantity of creosote oil injected into timber and piling by most of the timber-treating plants in the United States are much the same, and, in all the plants of which the writer has personal knowledge, depend upon the quantity of oil taken from a supply tank, as determined by the position of a float in the tank. This float is connected to a sliding indicator on the scale board in such a way that the depth of oil in the tank is recorded in feet and tenths.

The readings of the gauge taken during the treatment of a load are usually as follows:

A-Reading = Depth of oil in the supply tank before any oil is admitted to the treating cylinder;

B-Reading = Depth of oil in the supply tank at the instant the cylinder is filled; the oil taken from the supply tank equals the content of the cylinder when empty, less the volume of the timber load;

C-Reading = Depth of oil in the supply tank after additional oil has been pumped into the cylinder with the pressure pump; the additional oil forced into the cylinder equals the quantity of oil which it is calculated the timber load is to receive;

D-Reading = Depth of oil in the supply tank when the surplus oil is returned to the supply tank from the cylinder.

The difference between Readings *B* and *C* is the estimated quantity of oil the load is to receive. Lack of accuracy in the gauge mechanism is very objectionable, and should be reduced to a minimum; however, this defect is as likely to increase as to decrease the injection.

The lack of refinement due to the use of a measuring tank of large horizontal capacity might be avoided by having a supply tank of small diameter, say, 6 ft., to measure more accurately the quan-

* *Transactions*, Am. Soc. C. E., Vol. XLV, p. 498.

Mr. Lindsey. tity of oil forced into the cylinder by the pressure pump after the cylinder had been filled from the large supply tank; however, with the use of two supply tanks, it becomes more difficult to secure a satisfactory check on the quantity of oil used than can be ascertained when only one tank is in service.

The losses due to leaking pipes, valves and cylinder heads are extremely small in a plant where a proper degree of attention is given to the equipment. The loss due to leaking valves is the only one not readily observable, and close inspection of the condition of the valves should be made at regular intervals.

The quantity of oil absorbed by timber during the time the cylinder is filling with oil may be considerable, with well-seasoned stock, especially with such materials as paving blocks. After the *C*-Reading is recorded, and the pressure on the oil cylinder is released, preparatory to emptying the surplus oil from the cylinder, it is uncertain whether the entire quantity of oil injected into the load remains in the timber. To form a check on these probable inaccuracies, the *D*-Reading should be subtracted from the *A*-Reading, to determine the actual quantity of oil used. The difference gives the actual impregnation the entire load has received, provided there is no loss from leaking valves, pipes, or cylinder heads.

Underground supply tanks or dumping tanks are objectionable unless they are in a cellar and permit the inspection of all tank connections.

Where a plant is equipped with an elevated supply tank, the cylinder is usually filled with oil through a 10 or 12-in. pipe connection. The pipe from the tank to the pressure pump is usually from 3 to 4 in. in diameter. By returning the oil, by compressed air, after treatment, to an elevated supply tank through a 10-in. connection, the supply tank, the treating cylinder, and the connecting pipe lines could be fully examined by the inspector during the course of the treatment. With such arrangements, an intelligent inspector, after careful study of the equipment of the plant, should be able to keep a reliable check on the conscientious and intelligent operation of the treatment. This supervision would generally require day and night inspectors.

Any changes in the present methods which will place the creosoting of timber on a more precise and scientific basis, and afford the fullest possible opportunity for intelligent inspection, will be welcomed by all who have at heart the proper interests of the business.

It appears to be entirely practicable to place the inspection of air-seasoned stock less than 50 ft. in length upon a weight basis; however, in the treatment of green lumber, piling of any length, and air-seasoned material more than 50 ft. in length, the tank system of

measurement, by gauging the injection of oil, will probably remain in use. Effort should be made to abandon the use of underground dumping or supply tanks, and to simplify, as far as practicable, all oil-pipe connections between these tanks and the treating cylinders. The treatment which the material is to receive may be determined by the difference between the *A* and *D*-Readings of the gauge. Mr. Lindsey.

Quality of the Oil.—The presence of water in the oil should be carefully guarded against. None of the creosoting plants has stills or the oil manufacturer's proper equipment in order to free the oil from water entirely, and they have to depend largely upon the settling method. This consists in heating the mixture to, say, 180° fahr., then chilling it and allowing the water to float to the surface, where it can be discharged through a connection in the side of the tank. Steam is kept circulating almost continuously through the heating coils in the treating cylinders and the supply tank. It is of the utmost importance that these coils be tight, and that any leaks which may occur be closed promptly. The heating coils in the storage tanks, through which it is necessary to circulate steam when oil is to be drawn from the tank, should be examined from time to time, in order to avoid unnoticed leakage.

Oil received in barrels should be dumped promptly to avoid leakage. If the barrels are stored in the yard for some time considerable rain water will seep through the heads of the barrels.

The requirements that no oil with more than 8% of water be used, and that any excess of water between 2½ and 8% be compensated for by a proportionately greater injection of oil into the load, are reasonable, and should insure good work.

Specifications as to the quality of the oil are at present based largely on a distillation process. There is some difference of opinion as to whether it is advisable to allow a small percentage which will boil below 210°, or to exclude this light oil entirely. There is much difference of opinion as to whether it is desirable to have the distillate between 210 and 235° cent., 20% or 40%, or an intermediate percentage. It is certainly desirable to determine, as nearly as practicable, the most effective quality of oil necessary to preserve timber. The specification will be modified generally, however, by commercial necessities. After the coal-tar manufacturer has extracted from his tar all the higher-priced products, the residue or creosote oil is sold to the timber-treating plants. If the purchaser of creosoted lumber is informed that specifying a maximum distillate to 235° cent., of 25%, instead of 45%, will mean a 20% increase in the price of his treated timber, he is very apt to change his specification, feeling doubtful whether the increase in price is compensated for by the better quality of oil. The consulting engineer of one of the largest purchasers of creosoted timber in the

Mr. Lindsey. United States called for bids in December, 1904, to furnish the material needed by his client during 1905, and specified that the entire distillate, up to 235° cent., must not be greater than 30%, a quality of oil similar to that proposed by Mr. Stanford. Before the date of the letting of this contract the engineer advised all bidders that he had found it necessary to revise his specification in order to avoid excessive cost to his client. The revised specification allowed a maximum distillate of 60% up to 235° cent.

When the coal-tar manufacturer finds use for a portion of the present excess of oil boiling between 200 and 235° cent., it will be practicable, without excessive extra cost, to conform to the specification proposed by Mr. Stanford.

In determining the steam pressure and the length of the steaming period to be used in the treatment, the quantity of oil to be injected, as well as the size of the material, should be considered. According to the investigation of Dr. Hermann von Schrenk, of the United States Department of Forestry, the strength of creosoted material is affected, not only by the heat used during treatment, but also to some extent by the oil injected. The experiments indicated that an injection of creosote oil weakened the stick to the same extent as the impregnation of an equal quantity of water.

Mr. Stanford's observation, that heart wood is practically impervious to oil, probably applies mainly to long-leaf yellow pine piling, and particularly to the butt half of the pile, where the sap ring receives practically the entire impregnation. The writer has seen a section, from 10 to 15 ft. from the top of a long-leaf pine pile, completely saturated with oil. Where lumber is treated the impregnation in long-leaf heart pieces is generally from $\frac{1}{2}$ to $1\frac{1}{2}$ in. Short-leaf open-grain pine is best adapted to receive a satisfactory impregnation, and pine of this class should be secured for treatment if practicable.

The writer has an 8 by 16-in. yellow pine stringer, creosoted by Mr. J. W. Putnam at the West Pascagoula Plant in 1877, which was completely impregnated with oil at a section 3 ft. from the end of the stick. This stringer stood service in the West Pascagoula Bridge for more than 27 years. Such impregnation, in long-leaf yellow pine heart material, is unusual, however. Mr. Putnam describes the treatment of this bridge material for the New Orleans and Mobile Railroad in his letter to the Committee of this Society which, on June 24th, 1885, made a report on the preservation of timber.

It would be interesting to know if the plant, at which Mr. Stanford inspected the treatment of 80-ft. piling where such deficient treatment was secured by the tank measurement method of injection, was equipped to ascertain the actual quantity of oil used

ERRATUM.

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Page 31, Lines 10 and 11: For "The oil appears to be present only in the cell walls," read "The oil appears to be present only in the cell openings, and not in the walls themselves."

in treatment by taking the difference between the *A* and *D*-Readings. Mr. Lindsey.

W. K. Hatt. Assoc. M. Am. Soc. C. E. (by letter).—The form of Mr. Lindsey's statement, that the investigations by the Forest Service, United States Department of Agriculture, have shown that an injection of creosote oil weakens a stick to the same extent as the impregnation of an equal quantity of water, is somewhat misleading. Mr. Hatt.

The presence of the creosote oil does not directly affect the material of the cell walls. The oil appears to be present only in the cell walls. After wood has been steamed, the fibers are saturated with water (the ties having gained weight during the steaming process), the resins, etc., leached out, and the structure of the wood left in a condition such that the preserving fluid can be injected with less difficulty than would be the case in unsteamed wood.

The steamed wood is somewhat weaker than green wood. If the steam pressure is sufficient, the temperature will be high enough to scorch the wood, and unduly promote the process of disintegration of the cellulose. Or the duration of a given steam pressure may be sufficiently prolonged to produce this same result, which is conditioned by the state of the timber with respect to its seasoning, character of growth, and size.

The subsequent injection of creosote fills the cell openings with the oil, but the cell walls seem to be unaffected. That is, wood that has been steamed and then creosoted is not weaker than wood that has been steamed; any weakening is due to the steam. The only effect of the creosote is to retard the subsequent seasoning of the timber; and this may not be a disadvantage.

The authority for the foregoing statements will be found in a circular by the writer, now in press, published by the Forest Service, entitled "Report on the Strength of Treated Ties."

H. R. Stanford. M. Am. Soc. C. E. (by letter).—In reply to the question asked by Mr. Lindsey, it may be stated that, at the plant where the 80-ft. piling was treated, the equipment of the tanks and the piping connections made it possible to ascertain the quantity of oil which was used from the *A* and *D*-readings, but, because of the number of tanks and the complicated piping system, it is believed that the observation would have had but little value. Mr. Stanford.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1015.

THE CHANGES AT THE NEW CROTON DAM.*

By Charles S. Gowen, M. Am. Soc. C. E.

With Discussion by Messrs. William R. Hill, Frederic P. Stearns, Alfred Craven, George S. Rice, Edwin Duryea, Jr., and Charles S. Gowen.

In a former paper† the writer described at length the foundations and general plans and features of the New Croton Dam.

Since then, extensive changes have been made in the construction of this work. These were the result of the report of a Board of Expert Engineers, and were due primarily to recommendations made by the Chief Engineer, William R. Hill, M. Am. Soc. C. E., to the effect that the section of the dam at the south end, which was about 275 ft. in length, and known as the embankment section, and partly built with core-wall and embankment, be removed, and the masonry section of the main dam be extended to the south to replace it.

This change was authorized by the Aqueduct Commissioners in April, 1902, and was the occasion of considerable discussion in engineering circles at the time, and of some interest also on the part of the general public, as the questions of the increase in expense and loss of time in the completion of the dam were of importance, the latter especially, owing to the growing consumption of water and

* Presented at the meeting of January 18th, 1906.
† *Transactions*, Am. Soc. C. E., Vol. XLIII. p. 469.

the possible lack of an adequate supply before the dam, thus delayed in its completion, could be utilized.

It is not the purpose of the writer to dwell at length on the details of the discussion above referred to, which involved the general theory and practice in the construction of earthen dams. Nevertheless, it seems advisable, in view of the condition of certain parts of the work as found during the demolition of the core-wall dam, and of the attempt which has been made through these developments to justify and confirm the reports of the Board of Experts and the Chief Engineer, in the mind of the public and the profession, also in view of the general interest that has been occasioned by these changes, that the matter be presented from another point of view, and placed before the Society in the light of different opinions and conclusions.

The writer was Resident Engineer in charge of the construction of the New Croton Dam from its inception to within a few months, at which time the dam was practically completed, and, having resigned his position with the Aqueduct Commissioners, takes advantage of this opportunity to express himself upon a subject which naturally has been of interest, but concerning which, owing to his position, he has not hitherto felt at liberty to explain his views.

The New Croton Dam was designed originally by the late Alphonse Fteley, Past-President, Am. Soc. C. E., and its construction advanced under his direction as Chief Engineer of the Aqueduct Commissioners from 1892 to December, 1899, at which time he was compelled to resign owing to ill health. The general design of the various parts of this structure, the progress of construction up to January, 1900, and the various features of its foundations, particularly those relating to the full masonry section and to the core-wall and embankment section, were fully described in the writer's paper above referred to. This paper is illustrated with many sections, profiles and views, and the reader is referred to it should he wish for further information regarding the dam than this present paper, referring as it does to one particular subject, can afford.

The discussions following the report of the Board of Experts, in which the Board recommended the substitution of the masonry section for the core-wall section, were fully reported in the engineer-

ing press at the time, and the questions of the heights of core-wall dams and the functions and actions of core-walls and embankments were given in detail. In May, 1902, the work of tearing out the old section was started, and in March, 1903, had reached the base of the core-wall section in question, where it was found at one point that the limestone rock foundation was of a character to give rise, in the minds of some, to the question of its proper adequacy, and as to whether an error, either of judgment or neglect, had not been made in building the wall on it in the beginning. Mr. Hill has described these conditions as he found them, and has dwelt upon the disastrous results which, in his opinion, would have followed the completion of the dam on the lines as originally planned, in a paper published by the American Water-Works Association in 1905. The writer does not agree with these conclusions, and takes this occasion to review the subject.

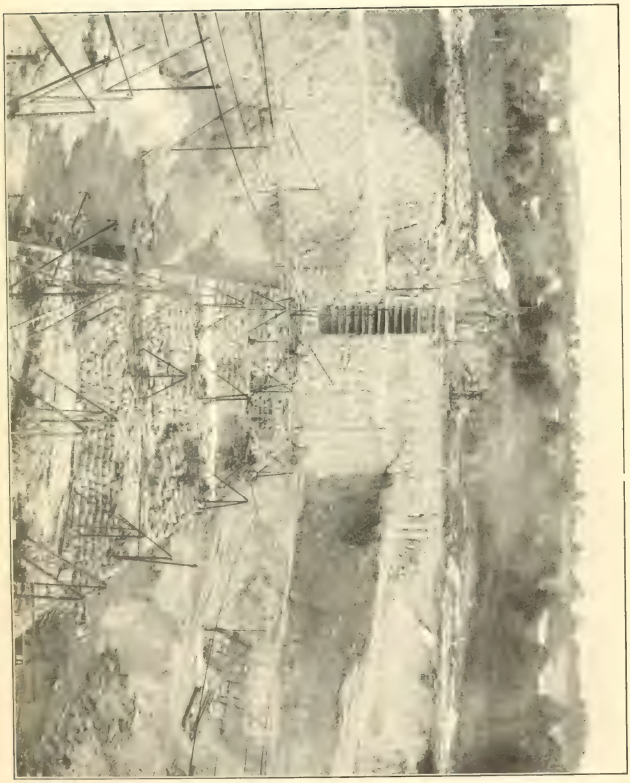
The following is a description of some of the illustrations used herein:

Fig. 1 is a plan showing in outline the section of embankment and core-wall lying between the south end of the masonry dam and its wing-wall and Gate-House No. 1, which section was partly finished and later taken out and replaced by the extension of the masonry dam.

Fig. 2 shows the profile of this part of the dam, with the original ground surface, the original rock surface, the rock surface as excavated for the core-wall foundation, the location of the dike of questionable limestone, the line of the top of the embankment as generally planned, etc. Fig. 2 also shows the cross-section of the embankment at the point in question, and, in section, the refilling, the undisturbed natural embankment, the core-wall trench, the core-wall as designed and partly built, and the embankment as planned along the so-called line of least resistance to the pressure and passage of water under or through the core-wall at the point in question.

This line of least resistance is located to accord with the definition given in the report of the Board of Experts, before referred to, and is shown on the plan by the lines, *A, B, C, D, E.*

Plate I shows the foundation masonry of the main dam, the hillside of hardpan into which this masonry was carried, and the trench for the core-wall.



FOUNDATION OF NEW CROTON DAM, SHOWING CORE-WALL TRENCH, HARDPAN SLOPES, ETC.

The Foundation.—As shown on the plan and on the profile, the seam or dike of questionable limestone extended from Station 1 + 30 to Station 1 + 95, a distance of about 65 ft. It was prepared under the engineer's direction to receive the core-wall base, and no question can be raised as to any oversight in connection with it. The trench

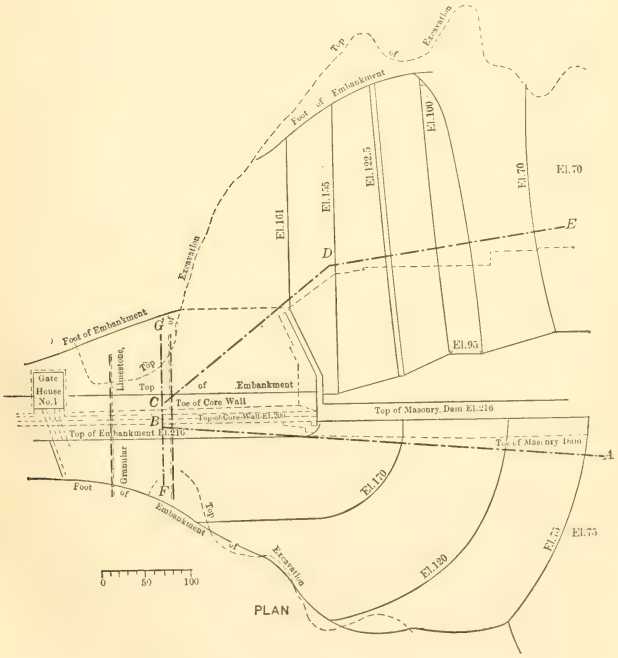


FIG. 1.

was sunk in the rock, varying in depth at different points from 4 to 12 ft. The floor thus formed was duly inspected, tested for bearing qualities, and then built upon. This floor, as shown at the time, was compact granular limestone, shaped with a pick with some difficulty,

and amply hard and solid to perform its duty, which was to carry the section of the core-wall above it.*

Upon the removal of the core-wall the rock showed much softer than it did originally. This was largely, if not wholly, due to the long exposure to which it was subjected during the progress of the excavation necessary to its removal, to weather, water, and the use of blasting powder. This resulted in relieving a compact compressed granular rock of its binding qualities, and rendered it more liable to the wear and disintegrating influence of the surface water. The assumption was made by some people that there may have been a gradual softening, or deterioration, of this bottom since the trench was opened originally, but there does not seem to have been adequate ground for such conclusions. In its characteristics, the dike was similar to, but more extensive than, other dikes found at various points along the line of the foundations of the core-wall and the main dam, from south of the location of Gate-House No. 1 to Station 7 + 50 on the main dam. The rock at all these points, if of sufficient bearing power, was assumed to be suitable to build upon, in view of the amount of embankment and refilling which it was planned to place on the up-stream side to seal the rock bottom, both along the main dam and the core-wall, from the water pressure on the up-stream side; and the dam as originally built has been allowed to stand upon it. The features of the limestone rock, forming as it does the base for most of the masonry section of the main dam and all of the core-wall section, were made, at the instance of Mr. Fteley, the subject of special investigation and report on the part of Professor Kemp, of Columbia University, in 1896, at which time a considerable portion of the main dam foundation had been uncovered. There is in this report no suggestion of any gradual deterioration of the limestone rock under any influences to which it might be subjected after the construction of the dam; on the contrary, the general tenor of the report is against such a theory, nor has the writer learned that later, in any report made by him, has Professor Kemp admitted that such changes were probable. In this connection it should be said that the whole question of building the dam on this limestone bottom was fully investigated and considered

* For a view showing this trench the reader is referred to Fig. 1, Plate XXXVII, Vol. XLIII, *Transactions*, Am. Soc. C. E.

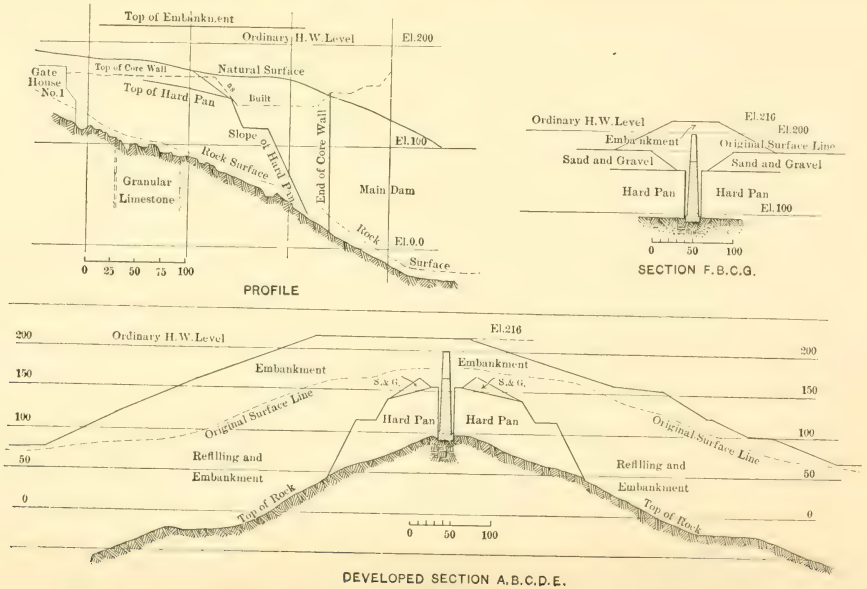

Top of Embankment
Ordinary H.W. Level
El. 200
Gate House No. 1
Top of Core Wall
Natural Surface
Top of Hard Pan
As Built
Slope of Hard Pan
El. 100
Rock Surface
End of Core Wall
Granular Limestone
Main Dam
Rock
El. 0.0
Surface
0 25 50 75 100
PROFILE
Ordinary H.W. Level
El. 216
El. 200
Embankment
Original Surface Line
Sand and Gravel
Sand and Gravel
Hard Pan
Hard Pan
El. 100
0 50 100
SECTION F.B.C.G.
200
150
100
50
0
Ordinary H.W. Level
El. 216
Embankment
Embankment
S. & G.
S. & G.
Original Surface Line
Hard Pan
Hard Pan
Original Surface Line
Refilling and Embankment
Refilling and Embankment
Top of Rock
Top of Rock
0 50 100
DEVELOPED SECTION A.B.C.D.E.

FIG. 2.

by Mr. Fteley. This is shown by the extensive system of borings, made and recorded before the beginning of the construction, as well as his reports upon the choice of location of the dam structure. The various features of this limestone rock were thoroughly understood, and the location of the south end of the main dam and the extent of the core-wall section were determined after a thorough study of the bottom and its overlying strata of gravel and hardpan. The mass of hardpan covering the southern slope of the valley and extending to the bed-rock from the end of the main dam was a controlling consideration in his acquiescence in the construction of the dam at this point, as the hardpan furnished an opportunity for the construction of an embankment section and for the exercise of a proper economy in the design of the structure.

The Core-Wall and Embankment.—At the location of this dike the core-wall, as shown on the profile and section, Fig. 2, was carried up to a height of about 75 ft. above the base and to within 35 ft. of its completed height. It was built in a narrow vertical trench, more than 60 ft. in depth, excavated in hardpan so hard that powder was used to get it out, and surmounted by a sloped trench reaching up from 25 to 30 ft. to the original surface of the ground which, at this point, is only about 20 ft. below ordinary high-water mark in the basin. Thus, at this point, a trench 80 ft. deep had been dug in the ground to found a core-wall for an embankment to retain 20 ft. of water above the ground.

This core-wall stood for several years, showing no signs of settlement, nor did a most careful examination of the wall and its foundations as it was taken down reveal any trace of settlement. At the top of the wall at various points along its length, and not confined to the short stretch covered by the dike in question, some temperature cracks had shown above the line of the refilling, and it was so evident that they were due to changes of temperature and, possibly, to some extent, to shrinkage of the setting mortar, that they were not given serious consideration until Mr. Hill's attention was called to them, and by him they were considered so serious that his first report and recommendation that the core-wall be removed were very largely based upon them.

It should not be lost sight of that this hardpan, forming the sides of the core-wall trench, extended to the rock, and was so compact

that a slope at the south end of the main dam excavation as originally planned and excavated, 100 ft. high and having slopes of ½ horizontal to 1 vertical, stood for several years without change in slope, except that due to scaling off in frosty weather, caused by the alternate freezing and thawing of its surface as it became water-soaked. This hardpan in its undisturbed condition is practically impervious to water, and surfaces which have been exposed to impounded water in the basin for 5 months of the present season stand solidly and without sign of change at a slope at least as steep as 1 to 1.

Fig. 2, Plate XXXVII, Vol. XLIII, *Transactions*, Am. Soc. C. E., shows this hardpan at the point at which the trench for the core-wall was later cut into the face of the slope which, at this point, as above stated, was ½ horizontal to 1 vertical. At the bottom may be seen a short stretch of the core-wall connecting the main dam masonry, which shows in the lower right corner, with the stretch to be laid in the trench when the cutting has been made. This slope extends, as shown, along the south end of the main dam and curves around the corners of the base, forming a vast pocket in the hillside into which the main dam section was carried to a horizontal depth of more than 200 ft. in the hardpan. Plate I is another and more extensive view of the hardpan slopes, and shows more clearly the horizontal depth to which the excavation had been carried into the hillside, as well as the vertical extent of the hardpan. This hardpan was a mixture of very fine sand with a small quantity of clayey material and boulders.

Fig. 2 shows a cross-section of the dam, normal to its line, at the point of questionable foundation, and the excessive depth to which the core-wall was sunk in the hardpan at this point, at which the water above the restored surface would be about 20 ft. in depth, would seem to be evident. In fact, it would seem that no practical consideration would warrant carrying the core-wall, at this point, more than a moderate distance into the hardpan.

Fig. 2 also shows a developed section at this point in the core-wall foundation taken along the so-called line of least resistance, as shown by the letters, *A, B, C, D, E,* on Fig. 1. On this section the bottom water level on the up-stream side is shown at Elevation 75, and the bottom land level on the down-stream side at Elevation 70. The

core-wall foundation is shown at Elevation 90 in the dike of granular limestone, and on either side of it is shown the extensive bank or wall of hardpan through or under which water would have to pass before reaching the wall. This is in addition to the very extensive and carefully made embankment and refill shown on the up-stream face above and below Elevation 75. On the lower side is shown the equally extensive refill extending to Elevation 70. The slope of this bank, on the up-stream side, is about 4 to 1, down-stream about $3\frac{1}{2}$ to 1. The thickness of the bank at ordinary high-water elevation is about 200 ft. and the thickness of the core-wall at Elevation 80, if the hardpan be taken into account and included, is about 300 ft.

It is difficult to understand, in view of the exhibit as above made, how any contention can be sustained that the granular or soft, or (even admitting it) the deteriorating qualities of the seam at Station 1 + 88, could have any effect upon the stability of the dam and embankment core-wall at this point.

The conditions of the rock base at this point were practically the same as at other points previously mentioned, under the core-wall and masonry dam, where the dam was allowed to stand as originally built, and the protection to these points from the up-stream refill in the case of the main dam, and from the refill and hardpan trench along the core-wall, is certainly no greater than would have been afforded in the case of the dike in question. It has thus happened that one seam has been dug out, though only partially, while, through force of circumstances, other seams no better protected have been allowed to remain. This seam was excavated to a depth of about 40 ft. below the first excavation for the foundation of the extended section of the main dam, and an adequate bottom for the foundation was obtained. This bottom is still of granular limestone, which, however, has materially diminished in width at the level reached.

The core-wall at this point would have been about 100 ft. in height had it been completed, and it would have been within the limits of good practice had the wall not been carried to bedrock. The core-wall was planned to be built along and up the valley slope on rock in order that it might be said that in this structure the continuity of the masonry and rock foundation is complete

throughout, and, as a matter of fact, it was not considered of importance that the foundation of this wall should be placed on rock, and the hardpan would have been entirely adequate.

In illustration of the above may be cited the Titicus Dam, where the core-wall is built on the hardpan for a height of more than 120 ft. in the north embankment; and in the south embankment the wall at a point of lesser height leaves the limestone foundation when it begins to dip below the horizontal and is built on hardpan for some distance until it reaches the limestone again as the latter rises above the horizontal.

At Carmel Dam the core-wall leaves the rock at a point where it is 75 ft. high, on hardpan, and at a short distance away the wall, 50 ft. high, is based on compact sand and fine gravel.

At Bog Brook the core-wall, about 70 ft. high, is built on a base of compact earthy material.

At Kensico Dam the core-wall is 64 ft. high, and rests on a hardpan base.

Fig. 3 shows the core-walls at Titicus, Carmel, Bog Brook, and Kensico Dams, their connection with the masonry section, and the points at which they leave their rock bases. As shown in the case of Carmel Dam, the core-wall leaves the rock at a point where the rock begins to dip, near the junction of the wall with the dam section; and such practice has not been uncommon in other cases of high dams, where the rule has been followed that a core-wall is adequately placed when founded upon a stratum sufficiently impervious and of sufficient extent to stop percolation. Numerous cases can be cited of core-wall dams varying in effective height from 100 to 120 ft., in different sections of the country, with core-walls based on compact earth or rock, in successful operation; and it is a matter of common knowledge to those interested that there is no record extant of the failure of an earthen dam with a core-wall due to filtration through or under the wall and the consequent movement of the down-stream bank. Failures have been due invariably to overtopping, to leakage along the line of some improperly placed pipe or sluiceway, or to the retention of water by the material of the down-stream bank.

Among the high core-wall dams above referred to are the following: The Druid Lake Dam, of the Baltimore Water-Works, which

has an effective height of 95 ft., with a puddle core-wall on a rock bottom 118 ft. below the top of the dam; the San Leandro Dam, near Oakland, California, with an effective height of 121 ft. and a puddle core-wall based on an earth bottom; the Tabeaud Dam, near Jackson, California, 123 ft. in height, with an effective height of 115 ft.; its core-wall being a puddle-wall on rock carried only part way up through the dam. The Temescal Dam, also in California, is from 95 to 100 ft. in effective height, and has a core-wall based on earth. There are many dams of great age in India, where for many years they have been used for irrigation purposes. One of these dams is 95 ft. in height, is built of clayey material, and, so far as known, has no core-wall.

In the foregoing statements the effective height is defined as the difference between the level of the water at high-water mark and the level of the point of intersection of the down-stream slope and the plane of the valley bottom.

To the contention that flowage is especially likely to occur along the face of the core-wall, between it and the embankment, it may be said that experience does not justify such a conclusion, and the case of the Titicus Dam may be cited in illustration. The conditions there, if anywhere, seemed to be especially favorable for such results, and where the core-wall leaves the rock (see Fig. 3), at a point about 300 ft. distant from the junction of the embankment cone and the up-stream face of the masonry wall, it is about 120 ft. high. For several years after the completion of this dam there was a gradual settlement of the up-stream bank, which amounted to 18 in. along this face, and the conditions would seem to have been especially favorable to develop a flow along the wall. There was, of course, a gradual compacting of the bank due to the settlement, but the Board of Experts, in their investigation, found no trace of seepage or flow in the down-stream bank or through the core-wall, and there does not seem to be any ground for the assumption that a flow developed through a core-wall, from any reason, would induce a flow along the side of the wall between it and the embankment, where, it must be remembered, the steady static pressure due to the weight of the saturated bank and normal to the assumed line of induced flow would act to prevent it.

This point is further illustrated in the case of the main dam

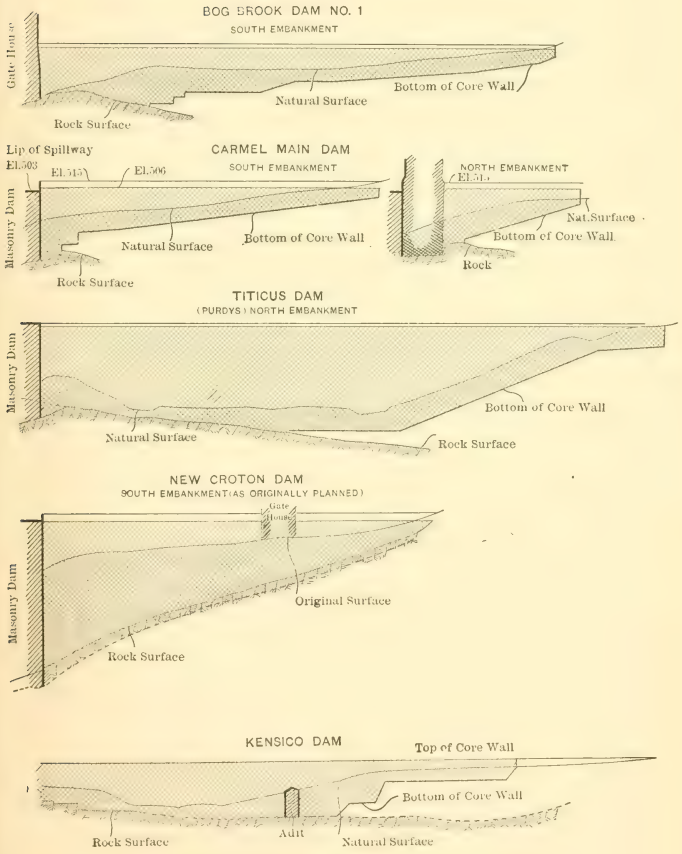

BOG BROOK DAM NO. 1
SOUTH EMBANKMENT
Gate House
Natural Surface
Bottom of Core Wall
Rock Surface
Lip of Spillway
El. 503
El. 513
El. 506
CARMEL MAIN DAM
SOUTH EMBANKMENT
Masonry Dam
Natural Surface
Bottom of Core Wall
Rock Surface
NORTH EMBANKMENT
El. 5[illegible]
Nat. Surface
Bottom of Core Wall
Rock
TITICUS DAM
(PURDYS) NORTH EMBANKMENT
Masonry Dam
Bottom of Core Wall
Natural Surface
Rock Surface
NEW CROTON DAM
SOUTH EMBANKMENT (AS ORIGINALLY PLANNED)
Gate House
Masonry Dam
Original Surface
Rock Surface
KENSICO DAM
Top of Core Wall
Bottom of Core Wall
Rock Surface
Adit
Natural Surface

FIG. 3.

section at the New Croton Dam, where the refill on the up-stream side stands from 100 to 140 ft. below high-water mark; and it is upon this fill that dependence is placed to stop flow through the fissured limestone which forms the base of the dam below.

The writer, therefore, cannot see that there is ground for the contention that the rock foundation of the core-wall was unsafe, nor can he, as he reviews the arguments on the general question of the substitution of the masonry section for the core-wall section, concede that the points were well taken. It would seem that the rejoinders to the reports, made by Messrs. Fteley and Craven in the engineering press, covered completely and fully all the points advanced, and disposed of the conclusions effectively, and that nothing further need be said beyond this: That the City of New York has expended unnecessarily nearly $1 000 000 and has failed to utilize at least two years of valuable time during which these changes at the New Croton Dam were being carried out.

DISCUSSION.

Mr. Hill. WILLIAM R. HILL, M. AM. SOC. C. E.—The proper plan and specification for a reservoir embankment is a subject susceptible of many conflicting opinions. The formulation of such a plan depends entirely upon the condition existing at the site of each particular structure. In the main, these conditions are: the character of the natural foundation, and the character of the earth available to make the embankment. With this information at hand, the question of the necessity of a core-wall arises, and, if it is required, its character and dimensions must be determined. Then, as to the embankment itself, its height, width, slopes, paving, and mode of construction must be fixed. To determine these important questions, the engineer must be guided entirely by his judgment, based upon experience and study of similar structures throughout the world. He might apply to science in vain, for he could get no help, as the efficiency of a reservoir embankment is not subject to computation; hence, it is not unreasonable to expect that conflicting opinions will arise as to the efficiency of a plan of a reservoir embankment.

Although this paper is entitled "Changes at the New Croton Dam," it treats of only one of the several changes that were made in the plan. To describe the structure briefly, it is composed of three distinct features, the spillway at the north end, the main stone dam, and the embankment with a core-wall at the south end. These, according to the original plan, had lengths of 1 000 ft., 600 ft., and 568 ft., respectively, making the total length of the structure 2 168 ft. Had the dam been built according to the original plan, the core-wall at the junction with the main stone dam would have had a height of 230 ft.

The first important change in the plan of this structure was made on September 16th, 1896, during the progress of the work. It consisted in extending the main stone dam a further distance of 110 ft., in substitution of the embankment and core-wall. That change was at once received with favor, and was carried out in the construction without any discussion whatsoever. It was of exactly the same nature and made for the same purpose as the change under consideration; that is, the main stone dam was extended in each case, with the sole object of reducing the height of the embankment and core-wall; and yet, while the first change materially increased the cost and delayed the completion of the work, it was not as effective as the change under consideration, as it resulted in reducing the height of the core-wall only 30 ft., still leaving it with the unprecedented height of 200 ft.

On January 1st, 1900, when the speaker assumed the responsibility for this work, the foundation of the stone dam had been com-

Mr. Hill. pleted to the surface of the ground, and the core-wall was completed, excepting the stretch under consideration, which lacked about 60 ft. of its height.

In the spring of 1901, the speaker's attention was called to five slight cracks in the core-wall, all within a distance of 100 ft. Relating to these, Mr. Gowen's paper states:

"It was so evident that they were due to changes of temperature and, possibly, to some extent, to shrinkage of the setting mortar, that they were not given serious consideration until Mr. Hill's attention was called to them, and by him they were considered so serious that his first report and recommendation that the core-wall be removed were very largely based upon them."

In reply to this, the speaker would state that he cannot concur in the opinion that the cracks were due to changes in temperature, as he could not expect contraction cracks to occur so closely together as five within a distance of 100 ft.; neither could he believe that they were caused by the shrinkage of the setting mortar, as such cracks could not extend through the wall, as they did in this case. But let the cause of the cracks be what it may, the cracks themselves were given importance by the speaker only inasmuch as they led him to a closer study of the plan, which study brought to light the really objectionable features, as shown by his report to the Aqueduct Commissioners, dated May 15th, 1901, wherein he reported that the core-wall was cracked, pointed out the objectionable features, and recommended that they appoint a committee of engineers to pass upon the adequacy of the plan. The following is quoted from that report:

"Even though there were no cracks, I consider that it would be unwise to complete the structure under the present plan, as I consider it would be an experiment."

Hence, the recommendation to remove the core-wall was not based on the existence of the cracks, but solely upon the opinion that the plan was inadequate. The concluding paragraph of that report is as follows:

"I make this recommendation after carefully studying the situation and plan, and I know that I am absolutely right, but, still I feel that it is due to you, as well as to myself, that we should be fortified by the opinion of three prominent engineers in this most important matter, and I respectfully ask you to take the necessary action."

The Commissioners, after personal investigation, agreed to this and appointed a committee of expert engineers, consisting of Messrs. J. J. R. Croes, Past-President, Am. Soc. C. E.; Edwin F. Smith, M. Am. Soc. C. E., Chief Engineer of the Schuylkill Navigation Company; and Elnathan Sweet, M. Am. Soc. C. E., former En-

gineer of the State of New York. The committee, after making an investigation, reported unanimously recommending the removal of the core-wall and the extension of the stone dam. Mr. Hill.

The general public will no doubt feel that great weight has been added to these conclusions by the concurrence of the eminent engineers, William H. Burr, M. Am. Soc. C. E., occupying the Chair of Engineering of Columbia University, and Nelson P. Lewis, M. Am. Soc. C. E., Chief Engineer of the Board of Estimate and Apportionment of the City of New York, both of whom had been asked by Mayor Low to investigate and report thereon. On April 16th, 1902, the Aqueduct Commissioners resolved to remove the embankment and core-wall and to continue the main stone dam.

Mr. Gowen's paper contains two general contentions; one, that the plan of September 16th, 1896, was adequate; the other, that the natural foundation of the core-wall was safe.

To take up the first contention, that is, of the adequacy of the plan. This paramount question is treated in a brief manner, and without giving a clear description of the part of the plan under consideration. The only cross-section accompanying the paper is one of the embankment and core-wall at a point about 170 ft. from the end of the stone dam. In reference to this point, it states that the original ground is only 20 ft. below ordinary high-water mark, and that the core-wall, 110 ft. high, was built in a trench 80 ft. deep.

The paper contains what he designates as a developed section, and this also passes through the core-wall at the same point, that is, about 170 ft. from the end of the stone dam, and, after passing through the core-wall, the section then follows on the so-called line of least resistance to the pressure and passage of water under or through the core-wall. This line is not a straight line at right angles to the structure, for it makes an abrupt angle on each side of the wall, both deflecting northerly; in fact, the up-stream line follows the top of the embankment to its end, and these two lines on opposite sides of the wall diverge from each other at an angle of only 45 degrees. On this crooked section, the author states:

"The thickness of the bank at ordinary high-water elevation is about 200 ft. and the thickness of the core-wall given at Elevation 80, if the hardpan be taken into account and included, is about 300 ft."

Here, it might be interesting to note that, on a true cross-section, the bank was to be 30 ft. thick at the top, while the wall at the elevation noted was to be only 17 ft. thick. The paper also states that, on the developed section, the slope of the embankment on the up-stream side is about 4 to 1. This is a mistake, as both the contract drawings and the plan accompanying the paper itself show the slope to be only 2 to 1.

Mr. Hill. Accompanying the paper is a plan showing in outline the section of embankment and core-wall between the end of the dam and the gate-house, a distance of about 275 ft., and a profile of the same showing the original ground and rock surface and the rock surface as excavated for the core-wall foundation.

Only two reasons are given to support the contention that the plan was adequate: One is a reference to a mass of hardpan covering the southern slope of the valley; the other is a denial of a statement that flowage of water is likely to occur along the face of the core-wall. What might be termed another reason is a citation of several dams that have been successfully built to heights ranging from 100 to 120 ft. As to the success of such structures, the paper contains the following:

"And it is a matter of common knowledge to those interested that there is no record extant of the failure of an earthen dam with a core-wall due to filtration through or under the wall and the consequent movement of the down-stream bank."

In reply to this the speaker would state that the records show that the Mill River Reservoir Dam, at Williamsburgh, Mass., burst on May 16th, 1874. It was an earthen dam, with a masonry core-wall 600 ft. long and 43 ft. high. Water found its way under the core-wall and destroyed the embankment. The reservoir was suddenly emptied into a narrow valley, causing the loss of 140 lives and the destruction of about $1 000 000 worth of property.

The author also makes a reference to two rejoinders to the report of the Committee of Expert Engineers, when, without giving any information as to the contents of those rejoinders, his deduction from them is that they would seem to have covered completely and fully all the points advanced by the expert engineers and disposed of their conclusions effectively.

The foregoing constitutes all the information contained in the paper, concerning the dimensions and general features of the structure, and the reasons to support the contention that the plan was adequate. Upon these, so far as the paper is concerned, is based the conclusion:

"That the City of New York has expended unnecessarily nearly $1 000 000 and has failed to utilize at least two years of valuable time during which these changes at the New Croton Dam were being carried out."

The speaker, before presenting his view regarding the stability of the plan, deems it necessary to give the following brief description of the part of the embankment and core-wall under consideration. It extended from the end of the stone dam a distance of about 275 ft. to a gate-house built in the embankment. The core-wall at the end of the stone dam, as before stated, was to have a height of 200 ft.,

and, at the gate-house, a height of 90 ft. The embankment was to Mr. Hill.
be 30 ft. wide at the top, with sides sloping in the ratio of 2 horizontal to 1 vertical. The lower portion of the inner slope, to a height of 16 ft. below the crest of the spillway, was to be paved with stone, 18 in. thick, laid dry, upon 12 in. of broken stone; and, on the upper part of the slope, to a height of 12 ft. above the crest of the spillway, the paving stone was to be 2 ft. thick, upon 18 in. of broken stone. The core-wall in the center of the embankment was to be 4 ft. higher than the crest of the spillway, 6 ft. wide at the top and increasing to 18 ft. at a depth of 136 ft., then it had the same width to the base. The high end of the core-wall had been built in a wide pit. That was a necessary excavation for the end of the stone dam, which was 164 ft. wide at the base, while the core-wall was only 18 ft. at its base. The slope of this pit extended southerly along the line of the core-wall for a distance of 150 ft.; thus the core-wall at its highest end was not built in a narrow trench below the surface of the ground, as is usual in ordinary cases. The outline of this great pit is shown on the plan accompanying the paper, and is marked "Top of Excavation."

There are in the plan three objectionable features which influenced the speaker to recommend the removal of the embankment and core-wall. They are as follows: First, the excessive height, narrow base, and unstable foundation of the embankment; second, the great height of the core-wall; and, third, the means afforded water to reach the core-wall.

To take up the first, the embankment: It was to be 150 ft. high, and only 650 ft. thick at the base. This section would be not only about 30% higher than any heretofore built, but, in comparison with other high embankments, its base was narrow for its height. As an example, the Amawalk Dam, which forms one of the upper Croton Reservoirs, while only about half the height, 85 ft., yet has a base wider than that of this embankment of unprecedented height; and, further, this embankment was hazardous because of the unstable nature of its foundation. It was founded over a great refilled pit, which was 360 ft. wide at the top, 170 ft. at the base and 70 ft. deep. This pit was a necessary excavation for the foundation of the end of the stone dam, which was 164 ft. wide at the base, as before stated. It would be impossible to refill this pit as compactly as the original hardpan; hence the safety of the reservoir was dependent not only on an embankment of a problematic section, but this problematic section rested upon an unstable foundation.

The second of the objections: The core-wall of this embankment was to have the great height of 200 ft. and with no lateral protection or support whatsoever from the original ground, as the artificially placed earth on each side of the wall in this wide pit had

Mr. Hill. the height of the wall itself, 200 ft. The natural hardpan would afford no protection whatsoever here, inasmuch as it had been excavated to its entire depth; in fact, the underlying rock had been removed to a depth of about 15 ft. Considering the height of the wall, and this in artificially placed earth, it could be but an experimental structure, inasmuch as it would be about twice the height of any heretofore built.

The third objection, the means afforded the water to reach the core-wall: This is another serious objection, as the water, by starting at the end of the embankment in the reservoir and following between the face of the stone dam and the embankment, would inevitably reach the core-wall. It would be impossible to puddle or otherwise compact the embankment against the dam to prevent this, as settlement would surely follow in any embankment of this great height, and the settlement of the material under the projecting parts of the rock-faced masonry would leave cavities for the passage of water. This objectionable feature here exists because of the combination of a stone dam and an embankment, while it could not exist in either a continuous stone dam or, on the other hand, a continuous embankment and core-wall.

A fourth objection might here be stated, namely, the permeable and light character of the earth of which the embankment was made. Relating to this material, the Committee of Engineers reported:

"It is permeable to water under any head from 3 to 150 ft., and, when exposed to the direct action of water, it disintegrates and assumes a flat slope, the surface of which is best described as slimy."

Thus it will be seen that the safety of this reservoir was dependent, not only upon an embankment made of permeable material and of a problematic section resting upon an unstable foundation, but also upon a core-wall of phenomenal height, unprotected and unsupported by original soil and attended with the greatest of all possible risks; that is, the means afforded water to reach the center of the embankment against the core-wall. Such a structure, in the speaker's opinion, cannot be regarded as anything but an experiment, as it is abnormal and unprecedented in all its dangerous features. Thus, as the speaker was thoroughly convinced that the plan was inadequate, he was left no alternative but to condemn it.

Before closing, the speaker wishes to state that he has no desire to discuss the contention that the natural foundation of the core-wall was safe, as he wishes to maintain the stand he took at first; that is, that the plan of the structure itself was faulty, without considering the physical conditions existing below the base of the core-wall, and that the modification of the plan has resulted in the completion of the structure in keeping with the report of the Board of

Expert Engineers, consisting of Messrs. J. J. R. Croes, Joseph P. Davis and William F. Shunk, who, in 1888, recommended that the Quaker Bridge Dam, for which this is a substitute, be a stone structure from end to end. Mr. Hill.

FREDERIC P. STEARNS, PRESIDENT, AM. SOC. C. E. (by letter).— Mr. Stearns. Mr. Gowen's paper presents in a very clear way the conditions surrounding the dike of questionable limestone found at the southerly end of the dam, and he gives convincing reasons in support of the view that the construction, as originally planned and, to a large extent, executed at that place, was entirely safe. The writer has never examined this limestone, but he has had occasion to make tests of the bearing capacity of other soft rock, using for the purpose apparatus copied from that used for testing foundations at the New Croton Dam, and was surprised to find what a great difference there was between a very soft rock and the hardest and most compact earth. Therefore, he would expect any rock found in this section of the country to support a weight equal to that of a masonry core-wall.

The permeability of a rock foundation, where the rock is of compact texture, depends upon the presence of seams or other passages for water, and not in any degree upon whether the rock is hard or soft.

In a case like that described, where the core-wall was built in a narrow trench cut in firm hardpan extending to the rock, and where there was also provided an embankment of fine clayey material of great dimensions, there must be taken into account the resistance of this earth to seepage and to water pressure.

It is quite often the case that an embankment built of earth containing a sufficient proportion of fine particles is as nearly watertight as a concrete or other masonry core-wall, but this is not recognized in all instances, possibly because the concrete has so much greater strength.

The Board of Expert Engineers, who recommended the changes at the New Croton Dam, caused many borings to be made in the embankments of the dams of the Croton system, and, in a majority of cases, the line of saturation determined by the investigations indicated no greater resistance to seepage or percolation at the core-wall than in the embankment of earth.

In view of the character of the earth at the part of the New Croton Dam under consideration and the great distance through the earth on the line of least resistance, the writer is of the opinion that the dam would have stood at this point without any core-wall, provided the up-stream part of the embankment of clayey material were carried down to join the hardpan, and that, with the core-wall as an added safeguard, this part of the dam would have had a greater factor of safety than the all-masonry section.

Mr. Craven.

ALFRED CRAVEN, M. AM. SOC. C. E. (by letter).—One should not be permitted to infer, by a perusal of the *Transactions* of this Society, that one of the greatest structures of its kind, designed and partially carried to completion by one of its most distinguished members, had been a failure to such an extent as to necessitate its partial demolition and subsequent reconstruction on different lines, without a full discussion of all the reasons that really brought about the change. Such a conclusion as to failure would undoubtedly be arrived at by one familiar only with Mr. Gowen's paper, "The Foundations of the New Croton Dam," presented on February 21st, 1900, on learning subsequently of the radical changes which have since been made.

Fortunately, Mr. Gowen has now supplemented his original paper by the discussion of the changes made, giving substantial reason why, in his opinion, they were unnecessary.

William R. Hill, M. Am. Soc. C. E., who was primarily responsible for the radical changes made, has replied to Mr. Gowen's later paper, giving reasons why, from his point of view, the changes were advisable; he has given his views, as heretofore frequently repeated in the technical journals as well as in the daily papers, and supports these reasons by noting their endorsement by a Board of Expert Engineers, and others.

It appears to be only proper, therefore, that the views, as to the changes in construction, of the engineer who designed this great work, the late Alphonse Fteley, Past-President, Am. Soc. C. E., should also have their place in this discussion.

Fortunately, Mr. Fteley's views are on record elsewhere. In *Engineering News* of December 12th, 1901, they may be found in full; herein will be noted such quotations, only, as bear particularly on the main questions involved. The writer also reviewed the subject at some length in *Engineering News* of January 12th, 1902.

Mr. Hill, in his argument on Mr. Gowen's recent paper, notes that Mr. Gowen has referred to the above-mentioned "rejoinders to the Committee of Experts without giving any information as to the contents of these rejoinders."

To avoid long technical descriptions and repetitions of arguments, and trusting somewhat to the reader's careful perusal of the papers mentioned, the writer has reproduced here, in Figs. 4 and 5, his own drawings from *Engineering News* of January 12th, 1902.

Mr. Fteley, after calling attention to the hardpan formation in which the core-wall was to have been built, says:

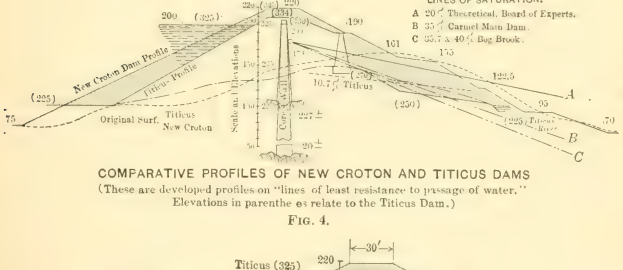
"In instances of this kind, when a rock foundation is found within accessible distance under the central part of a dam, it is very usual to abandon the rock foundation and to let the foot of the core-wall step up into the earthy materials of the side hill."

Mr. Craven.

He then cites the embankments of Bog Brook, Carmel Main Dam, Titicus "and many others" (see Fig. 3, page 43), and states:

"In the present case, however, the core-wall was extended down to the underlying rock into which a trench was excavated to receive the foot of the wall; this arrangement presents the additional advantage of establishing the high wall on an unyielding foundation.

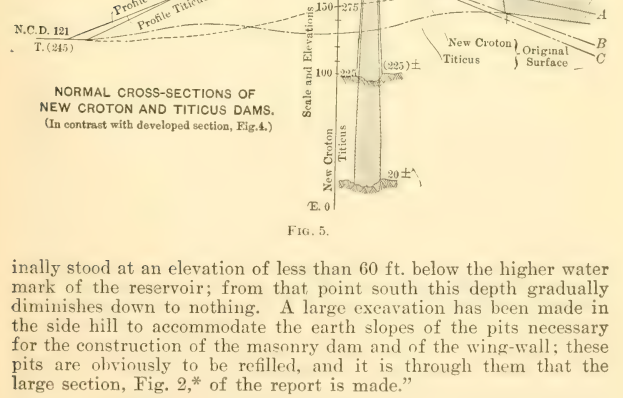
"At the southern end of the central body of masonry where the embankment begins, the height of the surface of the side hill orig-



COMPARATIVE PROFILES OF NEW CROTON AND TITICUS DAMS

(These are developed profiles on "lines of least resistance to passage of water." Elevations in parenthe es relate to the Titicus Dam.)

FIG. 4.



NORMAL CROSS-SECTIONS OF NEW CROTON AND TITICUS DAMS.

(In contrast with developed section, Fig. 4.)

FIG. 5.

inally stood at an elevation of less than 60 ft. below the higher water mark of the reservoir; from that point south this depth gradually diminishes down to nothing. A large excavation has been made in the side hill to accommodate the earth slopes of the pits necessary for the construction of the masonry dam and of the wing-wall; these pits are obviously to be refilled, and it is through them that the large section, Fig. 2,* of the report is made."

* Fig. 2 of the Experts' Report is referred to here.

Mr. Craven. Mr. Fteley, after taking exception to Fig. 2* of the Experts' report, which he very properly says "will convey an idea very different from what the facts will warrant," which indicates the refill of this pit as being a part of the earth dam, and which is furthermore taken so seriously by Mr. Hill, says:

"I may here point to the fact that, when building Titicus Dam, an extensive earth excavation was also made into the side hill on the north side of the masonry section in order to establish its footing and that of the adjacent core-wall on the rock foundation and, if a section of the structure were made on a similar 'line of least resistance' it would show the core-wall with earth embankments on each side, on a minimum slope of $1\frac{3}{4}$ to 1 with a height of 100 ft. A profile made on this 'line of least resistance' at Titicus Dam, in juxtaposition to profile No. 2,* would show a result in favor of the New Croton Dam."

The writer will here call attention to the fact that the Titicus * Dam, which will be frequently referred to, has been taken generally as a matter of comparison, as it approaches more nearly to the New Croton Dam in its general features than any other dam of composite type.

Mr. Fteley then considers the functions of an earth dam with core-wall, referring particularly to the case in point:

"Let us consider the two embankments of the dam separately, as they are called upon to act in a very different manner.

"The up-stream embankment will be in the water, and the (lower and larger part of it will repose, not against the core-wall, but against the main dam) from *A* to *B* (see plan herewith). When fluctuations occur in the reservoir, they will be so slow, on account of its vast area, that it may be said that no water will flow through the bank with sufficient velocity to displace any particles of earth; the only conditions left to be fulfilled are, consequently, that the embankment will be sufficiently water tight, and that it will not slough off or be washed off on the surface; the last condition will be met by covering the slopes as shown in the plans and specifications, with a layer of broken stone with heavy paving upon it.

"As to the condition of water tightness, it is sufficiently met, in my opinion, by the character of the materials to be used, which I have observed continuously for several years while refilling the excavation for the main dam.

"The up-stream bank of Titicus Dam, which is nearly 100 ft. high, with a similar slope, was built with materials finer than those used at Croton Dam, and, barring a slow and perfectly regular vertical settlement, which was expected, has acted in a very successful manner. For several years, whenever the height of the reservoir permitted, exact measurements were taken and the slope has never shown any mark of disturbance."

He then reviews the question of the materials used in the refill and embankment, as follows:

* Fig. 2 of the Experts' Report is referred to here.

"Before construction, both at Titicus and at Croton Dam, earth excavated from the places where it was expected to take it for refilling, was dumped in the stream, care being taken to form the dumps of the finest materials. In no case did those dumps, left unprotected, show a slope steeper than 1½ to 1. A bank, standing at repose in water, cannot be compared to such hill sides as the experts have observed in the valley where they were acted upon by ground water; if their comparison in that respect were correct, no slope of any kind would be practicable, and not an embankment of the dams built in the Croton Valley and elsewhere, under similar circumstances, would remain standing. Mr. Craven.

"As to the resistance of such an embankment to the percolation of water, it is obvious that absolute water tightness cannot be expected; and the experiments made at the Cornell Hydraulic Laboratory only illustrate that point; at any rate, it cannot be expected that the results of laboratory tests, on very small volumes of materials collected on the ground, can throw any valuable light on the ultimate behavior of an extensive bank through which the water would have to percolate for a considerable distance before reaching the masonry parts of the structure. Adequate knowledge of the materials used or to be used, and experience, must be depended upon to pass judgment on those matters; moreover, a comparison with the results obtained in the case of the dams built in other parts of the valley show that sufficient water tightness can be confidently expected from the proposed embankment."

The writer will remark here that the Board of Experts, while objecting to the quality of the materials used in this embankment and fill, states:

"All the tests indicated that this material, which we found to be almost identical in character with that which has been used in the construction of all the earthen dams in the Croton Valley, is permeable to water under any head from 3 to 150 ft., and that when exposed to the direct action of water it disintegrates and assumes a flat slope, the surface of which may be said to be slimy."

This statement (a portion of which is quoted by Mr. Hill), while evidently intended to be condemnatory of the material used, might well be considered, in view of the fact that of the ten earth embankments of the dams in the Croton Valley, all have successfully stood the tests from 10 to 25 years, as fully proving its unquestionable value in an embankment.

Mr. Fteley then reviews the functions and conditions for the down-stream embankment:

"The conditions under which the down-stream embankment would have to perform its functions would be entirely different. Nothing need be said of the water that may enter the bank from springs in the side hill or from the rain; the conditions in that respect will be the same as have always existed, with the difference that the turf on the surface will shed the greater part of the rain. There remains the water which will find its way through the

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masonry or through supposed deep fissures in the rock formation. As no water is expected to pass through the central body of masonry, the surface to be considered is limited to that part of the core-wall adjacent to the refilling of the excavations or to the embankment. What amount of water can find its way through the wall at that point can be appreciated from a comparison with the other dams mentioned in the report and from the comparative thickness of the masonry. In the majority of the cases referred to, the walls are of less thickness, and although the lower embankment will contain, as must be expected, a certain amount of water, the tests made by the experts indicate that a very small volume of it will flow through the wall. At Titicus Dam, where the wall has more thickness, the indications are that very little, if any, water finds its way through it; in the present case the wall, for the greater part of its height, at the points where the pressure is highest, is 18 ft. in thickness and built of excellent masonry. From these considerations the conclusion is consistently reached that, in view of the character of the up-stream bank and of the core-wall, a very small amount of water will reach the down-stream bank from those sources."

He then takes up the question of danger from saturation of the down-stream embankment, and questions the propriety of the arbitrary selection by the experts of the conditions in the outer bank of the Middle Branch Reservoir as a guide on which to base an arbitrary line of saturation—to govern in such cases, saying:

"The experts show a certain* 'line of bank saturation' as that likely to obtain in the present case. They base their statement on the observations taken by them at the various dams built in the Croton Valley; they find that the maximum safe height of an earth embankment with slopes of 2 to 1 would be 'on the bases of the loss of head and saturation at Middle Branch, 63 ft.; Bog Brook, 100.6 ft.; Titicus, 82.3 ft.; Amawalk, 72 ft.; Carmel Main Dam, 102.5 ft.' I fail to understand on what basis they state that from their observations the high embankment adjacent to the masonry dam would nearly approach the Middle Branch rate; such a conclusion would presuppose a complete knowledge of the comparative materials used, of the quality of workmanship, and of the various conditions existing during construction, which cannot now be obtained, as the Middle Branch Dam was built more than twenty years ago.

"Titicus Dam, with its high embankment and its heavy core-wall, is the structure to which can be best compared the New Croton Dam in several respects, and the experiments show that very little water, if any, finds its way through the core-wall, the water in the outer embankment standing 40 ft. below the reservoir level. A similar result is expected in the present case, and should a small amount of water find its way through the wall, the lower embankment, which is to be formed of comparatively porous materials, would allow of sufficient drainage, inasmuch as (to quote from the expert's report), 'the slope of the surface of the saturated earth in the bank is determined by the solidity of the embankment.'

* These lines of bank saturation are shown on Figs. 4 and 5.

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"The Auxiliary Carmel and Titicus Dams show very favorable results, although the lower banks were formed of fine materials, none others being found within reach; with comparatively porous materials they would have shown steeper slopes of saturation. I cannot see the truth of the statement that 'the more compact the material of which the bank is built, the steeper will be the slope of saturation.' With compact material, the sectional area of flow is larger below a given level than with porous material, and as the bank slope is one determining factor of the line of saturation, this line tends to approach the slope line. With porous material in a down-stream bank the slope of saturation is steeper and the area of flow less. Unless the water finds its outlet on the face of the slope of the embankment, the slope of saturation will also be regulated by the fact that it will reach the ground-water level at a point near the toe of the slope.

"At the New Croton Dam, the down-stream bank has a 2 to 1 slope for a depth of about 60 ft. below high-water mark, and the unusual width of the top of the embankment is equivalent to a flattening of the slope. Below this, the retaining slope along the down-stream face of the main dam (on the 'line of least resistance') has a general inclination of nearly 3 to 1.

"The experts suggest that the alleged lack of stability would be, to a large extent, overcome by flattening the embankment or by facing the lower slope with a revetment of heavy stone paving; these two suggested additions are, in my opinion, unnecessary; if I were to suggest an improvement to the present plan, I would recommend the drainage of the lower parts of the embankment; this work could be done on an extensive scale, at a comparatively trifling cost, with excellent results."

After discussing other features of the masonry dam, Mr. Fteley concludes as follows:

"Economy of design, when properly applied, is one of the main principles of engineering; it was undoubtedly given due weight in this instance, and it should not be departed from without the clearest demonstration that the proposed change is a necessity. It is not thought that the experts' arguments would produce that conviction on those experienced in the construction and maintenance of earth dams, and their determination of a probable line of saturation does not appear to be logically deduced from their observations of existing dams, or to be based on a sufficient study of the mode of percolation of water through fine materials."

Mr. Hill remarks:

"Although this paper is entitled 'Changes at the New Croton Dam,' it treats of only one of the several changes that were made in the plan."

In reference to other changes, Mr. Fteley remarks:

"The experts object to the original plan of the dam, which they call unjudiciously designed, on the grounds that no provision had been made to meet the contingency of a sudden and exceptional flow of water due to a cloud-burst or to other causes. The fact is

Mr. Craven. that the original design was especially devised to meet that condition. Although the surface of the reservoir, covering thousands of acres, is so large that it would have a great equalizing power, the contingency of a sudden flood causing the overtopping of the masonry dam was carefully considered. To that effect, the top of the embankment was kept much above the crest of the masonry and a large amount of rock from the excavations was ordered to be placed on the top of the filling, below the dam, to prevent a harmful disturbance of the surface. The contract drawings are not at hand, but it is well understood that they are of a general character and cannot be expected to show all the details of the work, but the connection of the top of the embankment with the crest of the dam is shown on Sheet 22 of the Report of the Aqueduct Commissioners of Jan. 1, 1897, and the necessary orders for the performance of the work were given.

"I learn by the report of the experts that changes have recently been made, one being the raising of the crest of the masonry dam. The reasons for that change in the plan are unknown to me. I agree that it is injudicious, as it destroys a feature which was considered very important. The calculations for the stability of the dam were made in view of the original elevation of the crest."

The experts themselves say:

"The masonry dam should not in any case be built higher than was originally designed. Such a change destroys the harmony and efficiency of the design which, having been scientifically determined, should be rigidly adhered to."

Still, Mr. Hill persisted in raising the masonry dam in spite of this admonition; in other words, the views of the Board were to be given weight only in so far as they agreed with his own.

The writer will now comment on the subject from his own point of view.

Unquestionably, the vital point to consider, and the one which should receive most serious thought, is the integrity of the downstream bank of an earth dam.

If formed of too fine material, the flow of water through the bank, assuming that some will pass the core-wall, will be retarded, and the upper plane of saturation will take a flatter slope than through more porous material, giving a greater area of saturation, and may eventually reach the outer slope of the embankment, causing a sloughing off of the material and thus endangering the structure. If this is guarded against, the bank is in no danger; hence, while the outer bank is secure, the dam is safe, even though there may be considerable settlement.

The experts, discussing the question of saturation, to which they properly gave paramount consideration, established an arbitrary and extreme line of saturation, which indicated that the foregoing condition of liability to sloughing of the bank might result.

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They concluded, however, that this assumed condition for the New Croton Dam "might be overcome to some extent by flattening the slope of the bank * * * so as to bring the probable slope of saturation not less than 10 ft. below the surface of the bank."

Mr. Fteley suggested, but did not consider it essential, that the same results could be attained by draining the outer bank. It seems clear, therefore, that either of these simple methods might have been followed, thus removing the slightest cause or necessity for the great expenditure of money and time which have resulted in the methods followed—the cost would probably have been less than $50 000, as against $1 000 000 actually expended—and the loss of time in completion, amounting to two years or more, would have been obviated. Why was this not done? The experts say "it would add largely to the cost and would disfigure the appearance of the dam."

In discussing the question of saturation, Mr. Fteley takes exception to the views of the experts wherein they contend that "the more compact the material of which the bank is made, the steeper will be the slope of the saturation."

The saturated portion of the dam is simply that portion below the inclined plane of the surface of the water in the bank, whether the material be ever so coarse or ever so fine; porosity is merely a degree of compactness or *vice versa,* and all bank material will absorb water.

Slope implies motion in water, and there is no absolute retention of water in the outer bank of a dam having its base below the plane indicated by the loss of head in passing through the inner bank and then through a further obstruction of either masonry or puddle. It is simply a partial retention, with motion through the bank, governed entirely by the degree of porosity of the material, and, unquestionably, the more porous the material in the bank, the steeper will be the slope at which water will pass through it. Just the contrary is claimed by the Board of Experts in their report.

Comparing the earth portion of the New Croton Dam generally with that of the Titicus Dam, referring to Figs. 4 and 5, and accepting the definition of the effective height of a dam embankment as "the difference between the level of the water at high-water mark and the level of the point of intersection of the down-stream slope and the plane of the valley bottom," it is evident that the New Croton embankment is only about 30 ft. higher than that of Titicus Dam, instead of twice as high. It is true that the New Croton core-wall has nearly twice the height of that at Titicus, but in the very nature of this case, where both walls are carried to the underlying rock, this difference in height is more apparent than real; more properly speaking, the difference is in depth below the base of the dams.

Mr. Craven. It will be seen by Fig. 4 that there is only 30 ft. difference in the embankment height: The New Croton has an interior slope of 2 to 1, Titicus of $1\frac{3}{4}$ to 1; the New Croton, on its outer slope, has a variable profile much increased in value by wide bermes carrying it far beyond the profile of the Titicus, which also has a variable profile abruptly broken by a wing wall, which was not intended as, and is not, a barrier to filtration.

This wall has only a shallow foundation, and, as will be seen by reference to Fig. 4, the slope of saturation for the Titicus Dam, as established by the experts, will pass under this wall and out through the restored surface beyond, which, in accordance with their theory, renders the bank unsafe.

The excess of the New Croton profile over the Titicus, as may be plainly seen, is due to its greater width at the water line; the New Croton having a width of 115 ft., the Titicus of only 72 ft. These measurements are on the delevoped "lines of least resistance." (The actual widths on normal sections are 110 and 74 ft.; see Fig. 5.)

This excess in the New Croton is divided between the outer and inner banks, and it shows largely in favor of the New Croton Dam, as was stated by Mr. Fteley.

Fig. 5 shows comparative sections of the two dams at right angles, or normal, to the slopes, taken, in each case, near the dividing lines between the earth and masonry dams. It represents truly the actual differences in sections, which differences, as in the case of the developed section, are unquestionably in favor of the New Croton embankment. It shows a dam which, by the definition of effective height, is only about 40 ft. high. This is just south of the heavy wing wall.

The developed profiles on "lines of least resistance" (Fig. 4) have been used in making comparisons only for the reason that they are a creation of the experts, and it is the only method by which the New Croton embankment could be made to appear higher than any of the dams now in successful service in the Croton Valley.

Assuming, however, that the outer refill is to be treated as a part of the bank proper, then, on Fig. 4, are produced the several slope lines of saturation as determined by the experts for the other dams.

The theoretical 20% line of the experts would indicate a danger point on the developed profiles, while the lines of Bog Brook and Carmel Dams indicate absolute safety, passing far below the 10-ft. limit of the slope of the bank, a limit of safety fixed by the experts, which in itself is rather an excessive requirement.

On the other hand, all the lines indicate unsafety in the Titicus profile. These lines are plotted with the experts' assumption of a loss of head of 17% of the depth of water in the reservoirs.

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The writer agrees fully with Mr. Fteley that there is nothing whatever to justify the adoption, by the experts for New Croton Dam, of the Middle Branch line of saturation as against the lines of Bog Brook and Carmel.

The experts admit that the core-walls and embankments of Amawalk and Middle Branch were not as carefully constructed as those of the other dams, yet they arbitrarily select the Middle Branch line of saturation to apply to the New Croton Dam.

For the purpose of showing his alleged "inadequacy of embankment" of the New Croton Dam, Mr. Hill compares the width of base and slopes with the broad base and flat slopes of the Amawalk Dam. It must be borne in mind that the Amawalk Dam embankment was made in the form of a great earth fill, no attempt being made to compact the material by rolling or ramming, but trusting to the great mass of material to supply the equivalent of more careful methods of construction. During the long period of construction the material was allowed to settle as best it would; therefore, it is not to be taken properly as a comparative construction with the dams built by the Aqueduct Commission where every precaution was taken that is essential in the construction of embankments, fully justifying the comparatively steeper slopes and more contracted bottom widths.

There is one point in Mr. Hill's argument to which he apparently attaches great weight; it is the alleged "unstable foundation of the embankment."

In articles which he has caused to be published elsewhere, he dwells as follows on this feature of the work:

"This embankment was hazardous because of the unstable nature of its foundation. It was founded over a great refilled pit (giving dimensions of pit). It would be impossible," he says, "to refill this pit as compactly as the original ground, hence the safety of the reservoir was dependent, not only on an embankment of a problematic section, but this problematic section rested upon an unstable foundation."

* * * * * * * *

"The water would be afforded freer access through the refilled material of the great pit than it would have in ordinary cases where the wall below the original surface of the ground is in a narrow trench and protected by the original soil."

* * * * * * * *

Also, "a fourth objection, * * * the permeable and light character of the earth of which the embankment was made, but even with the best material, an embankment so constructed would be insecure."

In other words, his contention is that a made embankment cannot be as solid as, and will permit the passage of water more readily than, earth in a natural state.

Mr. Craven. In his argument on Mr. Gowen's paper, he has repeated these statements in a somewhat modified form.

The well-known facts are, unquestionably, just the reverse of the above. A properly made artificial embankment, either puddled or rolled in layers, undoubtedly contains a greater quantity of material per unit of volume than an equivalent volume of the same earth in its original position, and is less pervious to water. This rule frequently leads to the removal of a considerable quantity of material, and to a refill, often with the same material, rather than build on the natural surface; and the writer, from his knowledge of the conditions at the New Croton Dam, cannot conceive why there should be in that case any exception to this generally accepted rule.

The fact must not be lost sight of that the material replaced below the lines of the natural surface of the ground is simply a refill confined in a great pit, and no amount of reasoning over this condition can make it an embankment.

This refill could be made, and in fact much of it had been made, to a height of about 100 ft. above the low point of the core-wall, with such care as to preclude the possibility of bringing any undue stress on the core-wall that would tend to its rupture, and, furthermore, when it was again finally removed, the wall was found intact.

It is interesting, therefore, to know (see both Figs. 4 and 5), that above this line of refill, while the dam is no higher than that at Titicus, the embankment, as pointed out by Mr. Fteley, is far more in excess, in its factor of safety, than that of Titicus; and the writer does not believe any one will question the adequacy of the latter.

Properly made embankments will not settle materially; in fact, several cases of dams of considerable height can be cited where accurate levels have been taken on the tops of the dams when completed, and again, long after the reservoirs had been filled with water, additional observations have shown absolutely no settlement of the earth banks. In cases, however, where settlement does occur, if due to the action of water, it must mean the displacement of so much water and a consequently greater compacting of the bank material.

In regard to percolation and filtration through sands and soils, it might be well to study the paper on the Bohio Dam, Panama Canal, by the late George S. Morison, Past-President, Am. Soc. C. E., presented before the Society on March 5th, 1902, and the discussion thereon by Frederic P. Stearns, President, Am. Soc. C. E., on the North Dike of the Wachusett Reservoir; they are instructive and also may be considered as pertinent to the subject under consideration.

The late E. Sherman Gould, M. Am. Soc. C. E., an engineer of extended experience in the construction of dams and reservoirs, has written:

"An earth embankment provided with a heavy masonry center wall carried down to a firm substratum, or, failing in that possibility, to a considerable depth below the surface, the depth being in inverse ratio to the compactness of the material, and well bonded into the sides of the valley, forms one of the best and safest dams which can be built." Mr. Craven.

It has further been remarked that:

"Of the materials used in the construction of dams, earth is physically the least destructible of any. The other materials are all subject to more or less disintegration or changes in one form or another, and in earth they reach their ultimate and most lasting form."

One more reason has been given, by Mr. Hill and the Board of Experts, why an earth dam should not be built more than 100 ft. in height, and that is a lack of precedent. If engineers wait for precedent surely no advance will be made in any form of construction. There is absolutely no reason why they should stop at 70, 100 or 200 ft. in height for an earth dam; it is merely a question of necessity or expediency. The profession of engineering and architecture would long ago have been at a standstill if "precedent" had been waited on. In this case, however, there is no lack of precedent; Mr. Gowen has cited several cases of earth dams, 100 ft. or more in height, which, in the opinion of the writer, are, all things considered, much bolder in their conception than the New Croton Dam.

Again, if "precedent" is to be sought to solve the question, it should be used negatively; if dams of more than 100 ft. in height had been properly built and had failed, a precedent might then have been established as against further efforts.

Had the test wells been made in the Middle Branch embankment 15 years ago; had the same results been obtained as now, followed by the same conclusions; had these conclusions been accepted, then it is self-evident that the Carmel, Bog Brook and Titicus Dams would never have been built, at least on their present lines; still, they stand to-day as masterful examples of good construction and contradictions to the theories and conclusions of the experts, and it is so with many other earth dams with no flatter slopes than are proposed for the New Croton Dam.

In conclusion, the writer desires to say that, in his opinion, there was absolutely nothing to fear from the completion of the New Croton Dam on the lines as laid down in the original plans, knowing full well the care with which these plans were developed and were being carried out.

George S. Rice, M. Am. Soc. C. E. (by letter).—The writer is pleased to note Mr. Gowen's careful consideration of the engineering questions involved in the changes lately made in the New Croton Dam. Mr. Rice.

Mr. Rice. The subject is one in which engineers, especially those employed by the Aqueduct Commissioners since the original investigations of the various sites, have been very much interested. Knowing the extreme care with which this and other sites were investigated, and having a knowledge of the conditions which existed there, as well as the immense amount of work done by Mr. Fteley in connection with the theory and design of high masonry dams, it is a source of regret to note the criticisms of, and the changes in, the plans originally contemplated for this great work. Mr. Fteley's experience in this class of work was probably greater than that of any engineer of the present time, he having been equipped, not only theoretically, but practically, with a knowledge of the subject.

The writer realizes that it is perfectly natural for the engineer, in considering the question of a dam of this nature, to assume instinctively that a masonry dam from one side of the valley to the other would be an ideal solution of the problem; but when one is familiar with the results of the construction of dams in America, more particularly in connection with such large works as the dams for the additional water supply of the City of Boston, and also the extensions in recent years for the water supply in the Croton Valley, the consummate skill with which this work was originally designed is perfectly apparent. This plan contemplated a masonry dam where the foundations were adapted for it, and when peculiar conditions were found at the southerly end of the Cornell site the subject was treated purely with an idea of meeting the conditions thus found.

In the design of this dam, Mr. Fteley's whole idea was to erect a masonry structure, where it was found better in an engineering way, but when, in his judgment, the conditions warranted, he used a core-wall with an earth dam, and showed in this the very best engineering, that is, good construction and economy. His judgment in this matter was based upon the nature of the material in the Croton Valley, with which he was fully acquainted, as he had designed and constructed several dams there. In the writer's association with Mr. Fteley, it was natural that he should have talked with him on this subject at various times, and consequently he was familiar with the reasons and purposes of his method in designing the dam.

In the last few years of Mr. Fteley's life it was a gratification to him to feel that the changes in the dam reflected in no way upon his judgment; he realized that these changes were made by those who did not have a full knowledge of the subject, and under the circumstances could not have had the experience, and that, therefore, the criticisms were illogical.

In the construction of this work Mr. Fteley had the advantage

of having assistants who were experts in this particular line; and if Mr. Hill, as Chief Engineer of the Aqueduct Commission, had consulted members of his own staff and followed their advice, he would have had the command of exceptional talent in this class of work, would have profited by such experience, and, in order to carry out his plans, would not have been obliged to go outside for engineering advice. In reference to certain questions relating to the aqueduct work, it has been noticeable that Mr. Hill followed the advice of his consulting engineers only when it seemed to suit his own ideas. Mr. Rice.

Mr. Hill, in his report to the Aqueduct Commissioners, in speaking of the changes, said:

"I make this recommendation after carefully studying the situation and plan, and I know that I am absolutely right," etc. etc.

Such an assumption, on the part of an engineer occupying Mr. Hill's position, would seem to show that he lacked a full grasp of this subject.

The writer agrees with Mr. Gowen that the changes which have been made in the completion of the dam were made at an extra cost to the city and a delay which would not have obtained if the original plan had been carried out, and, in his opinion, the dam as reconstructed is no better in carrying out its purpose than the earth section as planned in the original design.

Edwin Duryea, Jr., M. Am. Soc. C. E. (by letter).—The writer wishes to add his testimony, to that of Mr. Gowen, that the late tearing out of a portion of the embankment and core-wall of the New Croton Dam because of an assumed unsafe foundation was unnecessary from any reasonable considerations of safety. Mr. Duryea.

Questions relating to foundations are not amenable to exact or theoretical treatment, and their correct solution must depend on precedent and judgment. The natural conditions of foundations, both those used as precedents and those under design, are only imperfectly divided into classes—the classes often merging gradually into one another, without sharp lines of demarcation. Even exact descriptions of the natural conditions are difficult or impracticable to make, and successful precedents cannot be applied to new work without the exercise of much judgment.

Notwithstanding all this, and the latitude thus warranted for differences of opinion on matters relating to foundations, the writer believes that the fear of its safety which led to the tearing out of the embankment of the New Croton Dam was entirely unwarranted and was based on no sufficient reason.

It is undeniable that well-designed earth dams, with adequate provisions for spillways and for conduits, are as safe and as lasting as are masonry dams. Many old earth dams prove this to be true, even when only a puddle core-wall, or no core-wall at all, exists, and

Mr. Duryea. when neither dam nor core-wall is founded on rock; and many of these old dams are of great height. The use of a masonry core-wall is comparatively recent, and is almost unknown outside of America. The writer believes in it, and uses it, but regards it as an additional safeguard and insurance, generally advisable but seldom absolutely necessary; and, when it is used, there is no necessity to incur much additional expense to carry it to rock.

Since judgment plays such a predominant part in the design of foundations and dams, the personality and experience of the designer should carry great weight; and it is difficult to recall any engineer better fitted, by previous successful experience with dams, to decide such questions, than was the late Alphonse Fteley, Past-President, Am. Soc. C. E. When to his wide knowledge and experience were added his years of study and consideration, with the aid of an able corps of assistants, of the New Croton Dam itself, it seems that his mature judgment in this matter should far outweigh with the engineering profession the comparatively hasty judgment of his successor, William R. Hill, M. Am. Soc. C. E., and the Board of Experts.

Speaking of the structure itself, apart from its designer, the criticism of the writer, on that part of the embankment and core wall dam that was removed, is, not that it was in any degree unsafe, but, on the contrary, that in carrying down the core-wall through such a great depth of hardpan to rock of any quality, good or bad, the precautions taken were excessive and far above such as are usually considered necessary. This was probably recognized by Mr. Fteley himself.

It was a grief to the engineers who knew Mr. Fteley to see discredit thrown on his last and greatest work, and to think of the pain that this must have caused him. Mr. Gowen has performed a duty to the profession and to his memory in seeing that the *Transactions* contain a full statement of the unfounded nature of the criticism brought against Mr. Fteley's design for the New Croton Dam, and should have the thanks of the profession for so doing.

The writer wishes to add that he speaks with a full knowledge of the design and of local conditions, having been contractor's engineer on the Titicus Dam, and contractor's superintendent on both the Main and Auxiliary Carmel Dams; also having worked for about four months on the preparation of a contractor's bid on the New Croton Dam, and afterward visited the dam several times during its construction.

Mr. Gowen. CHARLES S. GOWEN, M. AM. SOC. C. E. (by letter).—In reviewing Mr. Hill's communication the writer finds it necessary to call attention to the fact that in the paper under discussion it is definitely stated that its particular purpose is to refute the statement made that the foundation of the core-wall was unsafe; accordingly, the

special object of the various sections and views submitted, together with most of the text, is to illustrate this point. The general question of the changes advocated by Mr. Hill was assumed by the writer to have been so thoroughly discussed in the engineering press that the reports and articles published are merely alluded to as justifying him in his contention that the changes were unnecessary. The above explanation seems to be called for, as Mr. Hill states that the writer makes two general contentions, the first of which is that the former plan of the dam (that of September 16th, 1896) was adequate, and he then proceeds to discuss and criticise, in this connection, the plans, sections and statements offered in reference to the second and main contention as to the adequacy of the core-wall foundations. Mr. Gowen.

Mr. Hill states that the only cross-section submitted is one of the embankment and core-wall at a point about 170 ft. from the end of the stone dam. Seemingly, he does not bear in mind that this section is to show the conditions at the most questionable point of the dike of granular limestone, concerning which the criticism of the safety of the core-wall foundation was made. He also forgets, apparently, in criticising the developed section, that this was made to illustrate conditions with reference to the same point in the limestone dike, and that it was made on the lines of least resistance (so-called) to percolation, in exact accordance with the plan pursued by the Board of Experts* in their report on the general question of the effective resistance of embankments and core-walls to percolation, and the resulting condemnation of the core-wall plan of the dam. He also says that it is a mistake to state that the up-stream slope of this embankment of the developed section is about 4 to 1. That there is no mistake about this, is apparent when it is noted that in this section the width of the embankment at the top is nearly 200 ft., most of it lying on the upper side of the core-wall line, which, with the slope below, as shown, is the equivalent of more than the slope claimed, particularly when the material lying above any ordinary slope line demanded at this point is taken into account.

Mr. Hill's citation of the Mill River Dam failure, in order to controvert the writer's statement regarding the causes of failure of earthen dams, calls for comment, and in reply it may be said that the report of the Committee of the American Society of Civil Engineers which was appointed to investigate this matter (Messrs. Francis, Worthen and Ellis) states, in view of the crude methods used in the design and execution of this work "it is obvious that this cannot be called an engineering work:" and also, "no engineer, or person calling himself such, can be held responsible for either its design or execution."

*See Report of Board of Experts, *Engineering News*, November 25th, 1901.

Mr. Gowen. In defining his position in the premises, regarding the general question of the changes made at the New Croton Dam, Mr. Hill states that it is not unreasonable to expect that conflicting opinions will arise as to the efficiency of a plan of a reservoir bank. Further on, he states that his recommendation to remove the core-wall was based on his opinion that the plan was inadequate, and he then, further, says in his report to the Aqueduct Commissioners recommending these changes, "I know I am absolutely right." Continuing, he instances the unanimous opinion of the Board of Experts supporting him, and the further concurrence of engineers in the employ of the city who were asked to give their opinions by the Mayor; and he accordingly concludes that, as far as the general public is concerned, great weight has been added to his conclusions.

In view of the foregoing, which, in the main, is a contention of absolute right by Mr. Hill, as to his position in the important matter of these changes, it is certainly proper to call his attention to the fact that this dam was originally designed after careful research and study, and its construction was supervised by an eminent engineer who had a long experience in the construction of high dams of various designs, and whose success in results has not been surpassed. A comparison of Mr. Fteley's professional record, in this respect, with that of any of the engineers whose dicta or opinions have been cited as bearing on these changes, would certainly warrant the claim that there is at least room for argument in this matter. A record of many years of notably successful professional work along certain special lines cannot be ignored, nor can there be assumption of absolute right in conclusions without at least prompting the query as to what qualification or authority there may be to warrant it.

As to Mr. Hill's contention that the first important change in the plan of the dam was made in September, 1896, before his accession to the position of Chief Engineer, and concerning his remarks in relation thereto, attention is called to this point, that this change was provided for in the specifications of the contract, and that the general plan defining the length of the main dam was tentative only. This change may be considered simply as a determination, of the Chief Engineer who originally designed the dam, of the position of the end of the masonry structure. This determination was reached after the information regarding the foundation material which the excavation afforded had been considered, and the principal motive for making it was the reduction in the height of the embankment above the restored or refilled surface. This height, accordingly, was reduced to about 50 or 60 ft. The question of the depth of the core-wall below the refilled elevation did not enter into the settlement of this question, except as it was incidental to the reduction of the

embankment height. It is perhaps well to say that this determination or change involved no question of taking down work already done, and consequent delay, and that all plans for the progress of the work on the dam from the beginning were made in view of the settlement of this question at the proper time. Mr. Gowen.

The extreme height of the core-wall at the point of junction with the main dam was 183 ft., including the depth excavated in the rock for its foundation, which was perhaps 15 ft. At a point 25 ft. south of its junction with the main dam the core-wall height was 166 ft. This is cited to illustrate the rate at which the extreme core-wall height decreased as the foundation rock slope rose to the south.

In considering the general arguments advanced by Mr. Hill in support of this change, it may be said that his description of the conditions planned at the junction of the main dam and the core-wall is practically correct. He neglects to state, however, that the embankment was to be carried to Elevation 220, thus giving it a thickness or width of more than 100 ft. at ordinary high-water mark, and, as above noted, his extreme height of core-wall is too great. The statement that, at the end of the stone dam, its width was 164 ft., is also excessive, 130 ft. being nearer the mark.

In his discussion of the embankment, he claims it to be 150 ft. high. Granting, for the sake of argument, that this refill should all be considered as embankment, if the Board of Experts' definition of its effective height be taken as being determined by the difference between high-water level on the up-stream side and the toe of the bank on the down-stream side, the height of bank due to ordinary high-water level would be 130 ft., and to extreme high-water level it would be 136 ft., this being on the line of the developed section. As to its thickness at the base, conceding, again, for the sake of argument, the refill to be bank, we must also follow the section of least resistance to percolation shown by the report of the Board of Experts, and we have for the embankment a thickness of at least 800 ft., in place of 650 ft., as claimed by Mr. Hill.*

A height of 136 ft. is not excessive, nor is 800 ft. an exceptionally narrow embankment base, even in comparison with existing structures. This is particularly evident when the comparison is noted of existing conditions between this proposed plan and the similar conditions at the Titicus Dam, shown in Mr. Craven's article in *Engineering News* of January 6th, 1902. Here it is shown that the effective height of the Titicus embankment is about 25 ft. less than that proposed for the New Croton Dam, and that the increased thickness and dimensions of the New Croton Dam bank more than compensate for its excess in height. This, therefore, in the light of experience, cannot to be a problematic section, nor does

**Engineering News*, December 12th, 1901. Plan in Mr. Fteley's report.

Mr. Gowen. there seem to be any proper question as to the stability of the fill described by Mr. Hill for the pit beneath this bank. This pit is a hole in the ground, the walls of which would surely retain the refilling, which would have been compacted by the tremendous weight of the refill and bank above Elevation 70, and to a degree at least equal to its original density; this, especially when it is considered that the compact hardpan taken from this part of the excavation did not extend to the bottom of the pit, but was underlaid by a thick bed of sand and gravel.

Mr. Hill lays stress upon the experimental character of the core-wall of 200 ft. height, having "no lateral protection or support whatsoever from the original ground." It is assumed that he refers to such support as would be afforded by the sides of a vertical trench sunk in the ground; but it is difficult to see what advantages there could be in the lateral support afforded to a wall in a trench refilled to a depth of 3 ft. or more on either side of the wall, compared with that due to a heavy refill or massive made bank carried up at equal heights on either side and compacted at least in its lower stretches by the great weight of the bank above. A refill, under such conditions, would certainly afford as much lateral support to any wall, however deep or high, as could possibly be afforded by the refill placed by hand in a narrow space between the face of the wall and the side of the trench, however carefully it was done.

The writer, therefore, cannot see that the depth of a core-wall below the elevation which defines the lower limit of the effective height of an embankment has any bearing upon the essential questions involved in this case, if such wall has a proper base.

As to the actual height of the embankment involved in this discussion, the writer cannot concede that that portion of the refill placed on both sides of the masonry dam at the south end below the original surface line has to be considered. The actual height of the embankment to be considered, that portion above the original surface line, was not more than 60 ft., and the refill below, forming, according to the claims of the Board of Experts and Mr. Hill, the remaining 76 ft. of the bank, should be regarded simply as a restoration of the general surface or topography. Experience with combination dams varying in effective height from 25 ft. to the height of the Titicus Dam, which so closely approximates at this point the New Croton Dam, justifies this conclusion.

For the same reason, *viz.*, experience with varying heights of combination dams, the fear that flow may take place along the face of a dam wall beneath the embankment is not warranted.

Finally, Mr. Hill refers to the permeable and light character of the materials used for the embankment, as stated by the Board of Experts, who are quoted as stating that it was permeable under any

head for 3 to 150 ft., etc. The writer feels free to say that he can- Mr. Gowen.
not understand the meaning of such a statement which might possibly be explained by the unquoted context. However, he has this to say of the embankment material; that it was composed of gravel, sand, sandy loam and clayey sand, properly mixed and compacted, and that it withstood such proper tests as the Board of Experts made, not excepting tests for permeability and slope action under water.

As to the assumption that Mr. Hill's course in changing the core-wall section of the dam was in keeping with the report of the Board of Engineers (Messrs. Shunk, Davis and Croes) who, in 1880, recommended an all-masonry structure for a dam at the Quaker Bridge site, it should be said that there is no proper parallelism to be cited, as the prevailing conditions at the sites of the two structures in question were different.

Mr. Stearns, in sustaining the writer in his contention as to the safety of the core-wall foundation, has referred to the investigations of the Board of Experts regarding the saturation of dam embankments, and to its conclusions that, in a majority of the cases investigated, the indications were that the core-wall offered no greater resistance to percolation than the embankment. If these conclusions are right, it would seem to be evident that the embankments approach the core-walls in density, and that, furthermore, there can be little or no pressure to act at the base of the wall and thus aid percolation under it. Such indications are certainly not prejudicial to the earthen dam as a safe structure.

Mr. Craven's discussion embodies the principal features of his own and Mr. Fteley's papers concerning the changes in the dams, which were published in the engineering press and referred to in the writer's paper. The writer fully concurs with Mr. Craven as to the desirability, under the circumstances, of bringing these matters into the discussion, so long as the general questions regarding the changes were renewed by Mr. Hill.

The comments of Mr. Rice, referring to his professional association with Mr. Fteley at the time the studies for the New Croton Dam were being made, and to his knowledge of the conditions under which the necessary investigations were carried out, are certainly of value, as they give additional weight to the general belief that Mr. Fteley's professional conclusions were based upon intimate and scientific knowledge of essential facts as well as upon broad views, and that they were accordingly reliable and abiding.

In closing, the writer fully recognizes the pertinence of Mr. Duryea's discussion, particularly on the point that Mr. Fteley, in carrying the core-wall to rock, felt that he was taking precautions which, under ordinary conditions, might have been considered un-

Mr. Gowen. necessary. The writer, however, has already dwelt upon this point, though not to the extent of quoting Mr. Fteley's views in the matter, with which he was well acquainted. He has stated that the core-wall was carried to rock through the hardpan principally for reasons of sentiment which it was thought should have consideration in connection with a project of so large and in some respects unprecedented a character. Mr. Duryea, very completely and in a few words, reviews the whole situation, so far as the changes are concerned, and gives additional weight to the testimony regarding Mr. Fteley's superior judgment and qualifications in connection with the design and construction of the New Croton Dam.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

Paper No. 1016.

A NEW GRAVING DOCK AT NAGASAKI, JAPAN.*

By Naoji Shiraishi, M. Am. Soc. C. E.

With Discussion by Messrs. E. P. Goodrich, M. Otagawa, Charles M. Jacobs, R. C. Hollyday, L. J. Le Conte, Charles Albertson, L. F. Bellinger and Naoji Shiraishi.

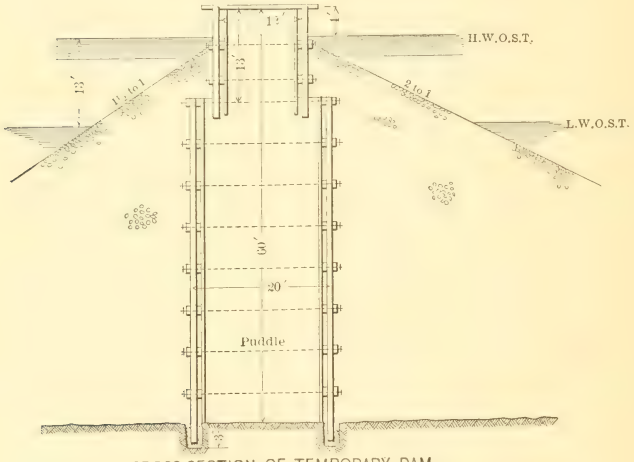
Dock No. 3 of the Mitsu Bishi Company's shipyard at Nagasaki has lately been completed. The giant steamer, *Minnesota,* 22 000 tons, was docked there just before her return passage to America. The dock will be the largest of the kind in the East for some time to come. Its principal dimensions are as follows:

Total length from inside face of outer shipgate to head of dock	722 ft.
Width of entrance at top	96 " 9 in.
Width of entrance at bottom	88 " 6 "
Width at top, dock proper	121 " 5¾ "
Width at bottom, dock proper	88 "
Depth at sill, below ground level	39 "
Depth at sill, below extreme high water	34 "
Depth of floor at head, below ground level	41 "
Depth of floor near entrance, below ground level	42 "
Height of keel blocks	4 "

* Presented at the meeting of December 6th, 1905.

The range of ordinary spring tides is about 10 ft., so that most of the common merchant vessels plying in eastern waters can be accommodated in the dock, even at low tide. Taking advantage of high tide, the largest modern steamers, such as the *Minnesota,* can easily be docked, as will be seen by reference to the dimensions given.

The rapid progress in shipbuilding has tended to increase the sizes of individual vessels in the West, but the lack of docks of corresponding sizes in Oriental harbors has prevented a similar



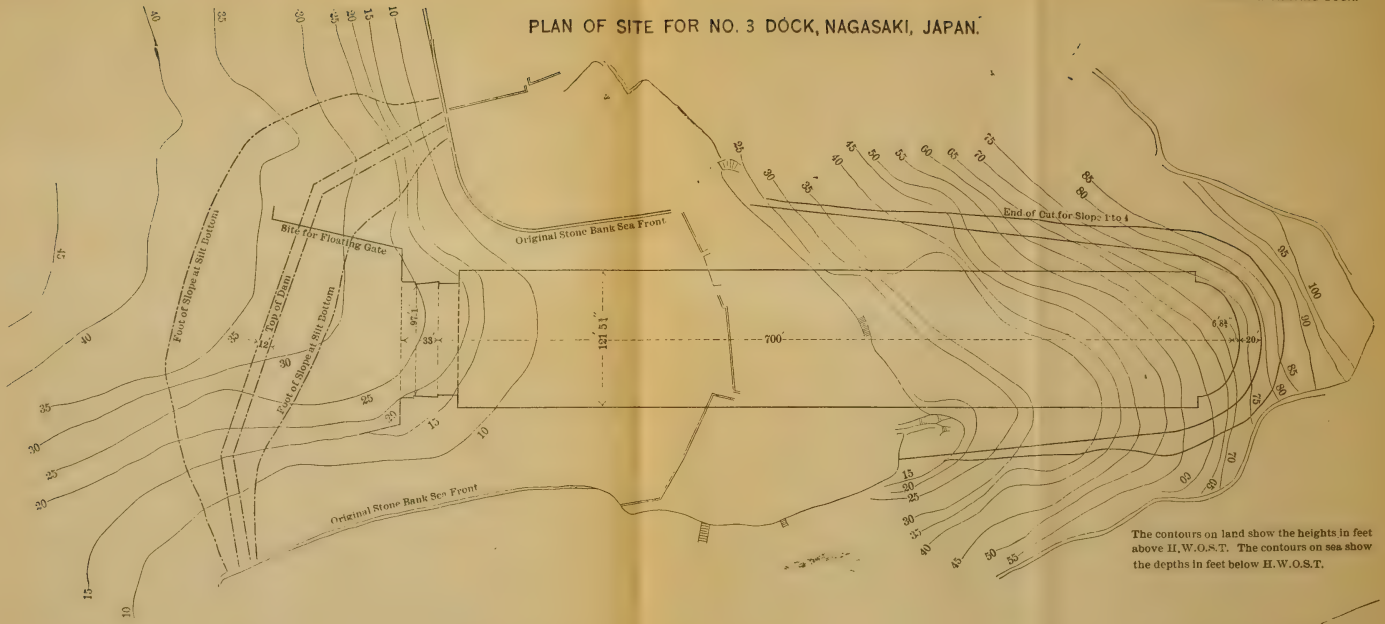
CROSS-SECTION OF TEMPORARY DAM.

FIG. 1.

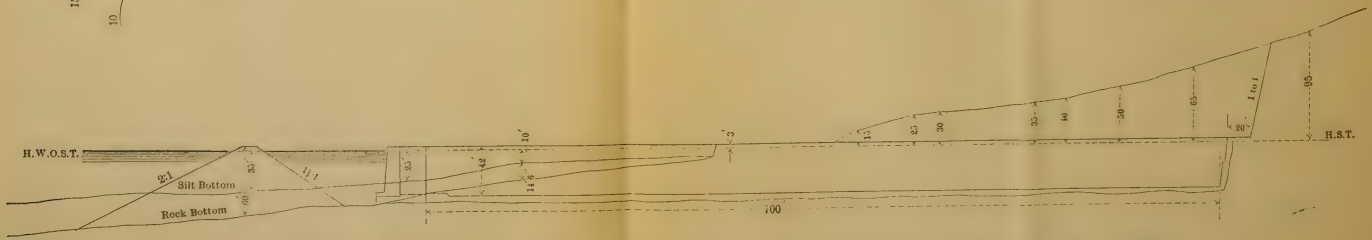
growth in the East. The Mitsu Bishi Company, in constructing the No. 3 Dock, had this in view, and wished to further this object rather than reap immediate benefit, from a business standpoint.

The configuration of the ground chosen for the site is shown on Plate II. The entrance was built in deep water, but the head was cut in a rocky bluff. The foundation is of andesite rock, and the concrete sub-foundation and cut-stone paving were laid directly on this firm base. Pile foundation was resorted to only for the

PLAN OF SITE FOR NO. 3 DOCK, NAGASAKI, JAPAN.



The contours on land show the heights in feet above H.W.O.S.T. The contours on sea show the depths in feet below H.W.O.S.T.



LONGITUDINAL SECTION OF DOCK SHOWING ITS RELATION TO NATURAL GROUND SLOPE

right-hand wing wall, where the rock stratum slopes down deep into the water.

The temporary dam for excluding sea water from the dock site is shown in plan on Plate II, and in section by Fig. 1. The clay puddle enclosed between two rows of sheet-piles was 20 ft. in thickness for the lower main part, but was reduced to 12 ft. near the top.

The debris from the rock cutting of the dock site was thrown outside the sheet-piles. The ground for the dam was prepared first by dredging the silt, with a Priestman's dredger; a narrow trench was cut in the rock under water to receive the main piles for fixing the sheet-piles. When the main piles were set in position, they were fixed in the ground by filling the trench with concrete. To prevent the percolation of water at the junction between the base of the dam and the natural ground, as well as to take precaution against the sliding of the dam, a concrete mound about 5 ft. high was built under water, along the inside row of the main piles.

After removing the silt layer, the depth of water near the middle of the dam was found to be more than 50 ft. at high tide, so that great care was taken, in building this dam under water, to make the puddle clay impervious at this great depth and correspondingly great water pressure. Fortunately, the execution of the work proved satisfactory, after pumping out the water.

The quantity of side cut and excavation was 256 000 cu. yd. The first 3 or 4 ft. from the surface was earth, but the remainder was of seamy andesite rock. It was first proposed to use rock-drilling machines, but the cheapness of manual labor, and the conditions being such as to permit of employing a great number of hands, induced the contractor to resort to the old method of drilling by hand. The average cost per cubic yard was 1 yen, including drilling, blasting and the subsequent removal of the debris. Some idea of the method of cutting and removal will be obtained from Plate IV, and reference thereto will be found further on in the paper.

For about one-third of the total length of the dock, near the entrance, the side walls were quite massive, the width at the bottom being 50% of the height, but for the remaining two-thirds of the length a thickness of only 4 ft. of concrete, with 2 ft. of cut-stone

masonry, was considered sufficient to line the rock cutting. In the vicinity of the sill site, any deteriorated rock surface left after excavation was carefully removed, and trenches were cut to receive the concrete sub-foundation, so that in some portions, just under the sill, for instance, the concreting is 14 ft. thick. For the main floor, however, the concrete layer upon the cleaned rock surface is only 2 ft. thick, upon which andesite cut-stone paving, 1 ft. thick, is laid. Cut stones are used only for the facing, coping and stepping, their backing being of concrete, so that, although the dock is, to all appearance, of cut-stone masonry, the main portion of the material is concrete. The facing near the entrance, and the coping and stepping are of granite, but the other parts of the masonry lining are of a cheaper material, andesite, and, to a careless observer, the dock seems to be granite throughout.

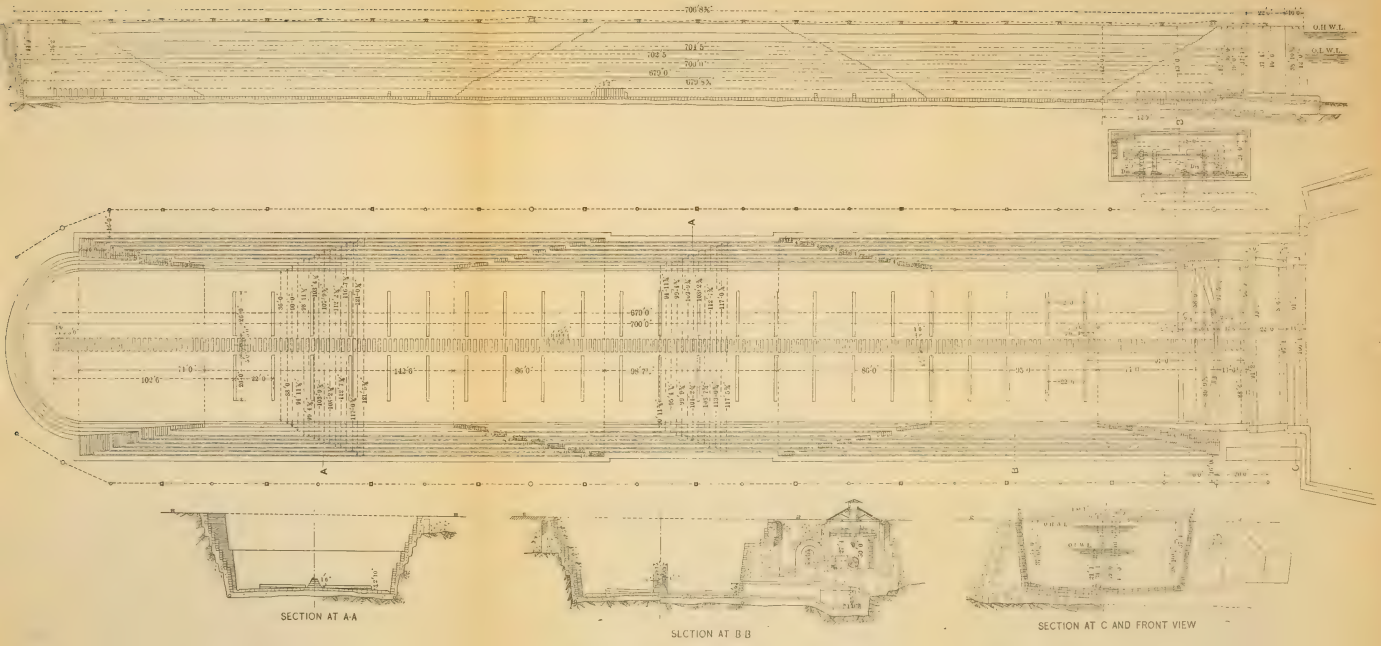
The concrete used for important parts was of the following proportions: Portland cement 1, sand 2, gravel 4; but, for the most part, the concrete consisted of: Portland cement 1, puzzolana 1, sand 4, gravel 8.

Puzzolana, which was conveniently obtained from Goto Islands (off the coast of Nagasaki), was found to be of considerable advantage, both in reducing the cost of construction, and in improving the imperviousness of the concrete. The wing walls are of concrete blocks made dry, and sunk in position. The proportions of the ingredients for the concrete blocks were: Lime 0.25, puzzolana 1, Portland cement 1, sand 4, gravel 8.

The pump-house is 24 by 76 ft., with its floor 27 ft. 4½ in. below ground level, or 11 ft. 7½ in. above the main floor of the dock. The sump is 9 by 52 ft., arched with bricks, and with its bottom 22 ft. 11½ in. below the pump-house floor. The space between the arch and the floor is one solid mass of concrete which resists the upward pressure of water by its dead weight. Four cast-iron suction pipes, each 33 in. in diameter, pass through this thick concrete. Four sets of 33-in. centrifugal pumps, suitable for direct coupling to electric motors, were supplied by Messrs. Gwinnes, Limited, England. The four pumps, together, are capable of discharging 16 000 000 gal. from the dock in 3 hours, when driven by the motors, each giving out 180 b. h. p., at a speed of 230 rev. per min. Under ordinary conditions, three of these pumps are used, the fourth being kept in re-

PLATE III.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVI, No. 1015.
SHIRAISHI ON
NAGASAKI GRAVING DOCK.

No. 3 DOCK, NAGASAKI DOCK YARD, MITSUBISHI CO.



serve. One 9-in. pump for drainage purposes, and one 6-in. charging pump, both mounted on the same bed-plate, are arranged to be driven by a belt from one motor.

Motors for the pumping plant, from Messrs. Siemens Brothers, are to work on a 220-volt continuous current. The current is taken, by wiring encased in earthenware pipes, laid underground, from the power plant common to all the workshops.

The gate, which is of the ship-caisson type, was designed and built at the company's own shop. When out of gate, the caisson is moored alongside the left-hand wing wall, which, accordingly, is vertical. Figs. 1 and 2, Plate IV, show the work during excavation, the first is a view of the work near the head, and the second near the entrance of the dock. The working area was wide enough for drilling about twenty holes for blasting at one time. In forming the steps to receive the side-wall lining, blasting was forbidden, in order to avoid loosening the remaining rock surface. The debris was removed by baskets on shoulders, and by hand-cars on light rails. When the excavation had been made to a considerable depth, a portion of rock was left at the middle of the dock to serve as an incline, and a winding engine pulled the hand-cars up to the surface of the ground, as shown by Fig. 1, Plate IV. In this photograph, the escaping steam indicates the point at which the drainage water was pumped. In Fig. 2, Plate IV, the mound of broken stones, upon which a group of men is standing, is the temporary dam erected to exclude the sea water from the dock site. A number of women were employed on this work. Plate VII shows the dock completed, and a few torpedo-boats docked. The common method of executing earthwork in Japan is shown by Figs. 1 and 2, Plate IV. It combines the use of machines with the old method of shovels, picks and baskets.

The innovation of using modern excavators, locomotives, machine drills, etc., may be suggested. No doubt the use of machines becomes economical when the amount of work is large enough to make the unit cost (including the machine's first cost) less than cheap manual labor; but, when one considers the local circumstances of the delay, the increased cost of importing the machines, the time required to train men in their proper use, and, further, the exceedingly cheap labor, say, 40 or 50 sen per day, and the small amount of

work to be done, there is not always an advantage in the use of machines. This was the case in the excavation of this dock.

Ever-increasing knowledge, however, as to labor-saving machines, the increase in wages, and for larger undertakings, will tend to bring Japanese methods more in accord with modern improved western practice.

The estimated cost of construction for the dock proper at the beginning of the work was as follows:

Temporary dam, together with its removal.	135 000 yen.
Side cutting	90 000 "
Dock excavation	120 000 "
Cut-stone masonry	335 000 "
Concreting	263 000 "
Building sheds	21 000 "
Pumping during construction	11 000 "
Timber works	22 000 "
Puddling back of side walls	6 000 "
Tools	25 000 "
Sundries	72 000 "
Total	1 100 000 "

The unit costs of the principal items paid to the contractors were as follows: Rock cutting, including its removal, 1 yen per cu. yd.; cut-stone masonry (granite), 30 yen per cu. yd.; concrete, 5 to 8 yen per cu. yd., according to the proportions of the ingredients; Portland cement, 4½ yen per bbl. Including the costs of the wing walls, pumping station and shipgate, in addition to the above items, the construction cost amounts to less than 1 400 000 yen, or $700 000 in American gold, which is an economical sum for such a dock, considering its dimensions and the quality of the finished work.

Charges due to the cost of land, electric power plant, traveling cranes, and other equipment, which should be included in the total cost, when the dock is viewed in its complete working condition, are here left out of consideration, as these accounts are not yet quite settled. The work was practically commenced in October, 1902, and was completed in March, 1905.

FIG. 1.—VIEW NEAR HEAD OF NAGASAKI GRAVING DOCK DURING CONSTRUCTION.

FIG. 2.—VIEW NEAR ENTRANCE OF NAGASAKI GRAVING DOCK DURING CONSTRUCTION.

DISCUSSION.

E. P. Goodrich, M. Am. Soc. C. E. (by letter).—This paper is of interest because it adds another to the international collection on this subject which was presented before the St. Louis Engineering Congress. Mr. Goodrich.

It is of further interest because it shows that, in almost every item, this dock has been designed according to the most advanced engineering knowledge.

In size it ranks among the large docks of the world. The wide entrance, approaching the 100-ft. standard, toward which modern docks seem to be tending, is of especial interest. The depth and length, of course, cannot be curtailed so easily when a dock is designed for use by modern shipping.

The endeavor to provide impervious concrete is of interest, because of the effect of the water in percolating through the concrete and disintegrating it to a perceptible extent. This has been encountered by the writer in the construction of several dock walls. Especial care should be taken with regard to this point, and the backs of walls should be given an especially impervious coating of mortar, or even water-proofed in certain locations. With the use of quartz gravel, the only material which is subject to disintegration by water is the cement, but great care should be taken even with this.

The design of the pump-house, as largely a subterranean structure, is not common, though two of this kind* were built at the New York Navy Yard during the writer's connection with it as Civil Engineer. The idea, in the design given in this paper, is identical with that at the New York Navy Yard.

Only three docks in the United States, as far as the writer is aware, are founded in whole or in part on rock, *viz.*, those at Halifax, Nova Scotia; San Francisco, Cal. (for the San Francisco Dry Dock Company), and at Portsmouth, N. H.

The costs of these three docks, for comparison with the Nagasaki dock, were as follows:

Location.	Date when completed.	Length.	Depth of sill.	Cost.
Nagasaki.....	1905	722 ft.	34. ft.	$700 000
Halifax.....	1889	600 "	30. "	750 000
San Francisco	....	750 "	32.5 "	474 000
Portsmouth.....	1904	750 "	30. "	1 087 956

It is of further interest to compare with these the cost of the celebrated Toulon dry dock, founded on caissons:

Location.	Date when completed.	Length.	Depth of sill.	Cost.
Toulon.....	1898	584 ft.	30.7 ft.	$1 003 000

*An interior view of one of these, together with discharge data of the pumps, etc., will be found in *Transactions*, Am. Soc. C. E., Vol. LIV, Part F, p. 429 *et seq.*

Mr. Goodrich. The unit costs of the various items of work on the Japanese dock are of great interest, as they show the effect of the cheap national labor. As the author remarks, it would hardly seem to have been economical to have tried to introduce machinery.

Mr. Otagawa. M. Otagawa, M. Am. Soc. C. E. (by letter).—There is an old Japanese saying that the light from a light-house reaches to a distance rather than to its base. In that way, the writer is sorry to say that he is not familiar with the new graving dock at Nagasaki, built by his friend, Dr. Shiraishi; in fact, he has not seen this dock, but has seen some graving docks in the United States. Therefore, not being familiar with the circumstances under which this dock was built, he cannot very well give a proper criticism as to its design and construction.

As to the cost of the dock, which, in spite of cheap labor in Japan, amounted to 1 400 000 yen, the writer believes that the cost would have been more than 50% less if it had been built 10 years earlier, or prior to the Chino-Japanese War, and has no hesitation in saying that the Russo-Japanese war made the cost still higher.

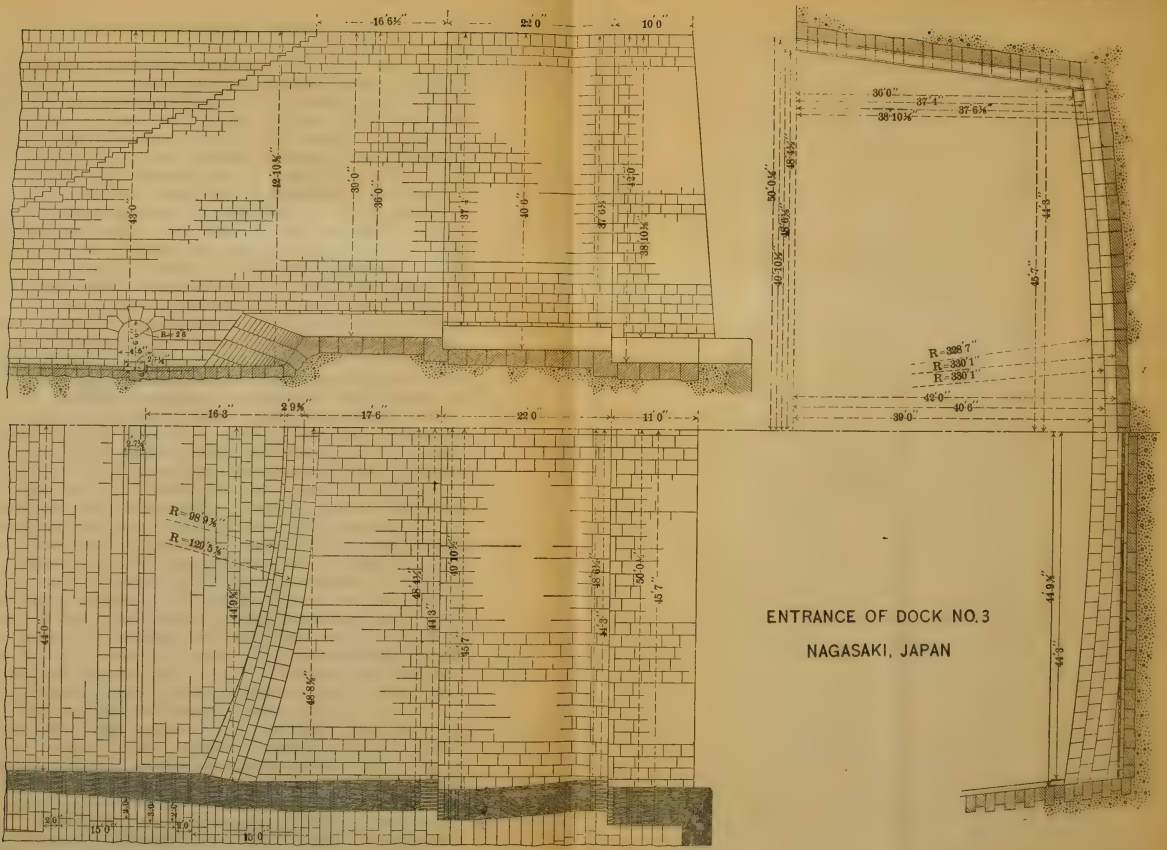
In proportion to the number of war ships and merchant steamers in Japanese waters, there are but few large docks in the Imperial Navy Yards and in commercial ports, and Japan is managing with fewer hospitals for her vessels than are other countries; but in the near future, additional docks of this kind must be built, and therefore the paper will be specially interesting for future reference.

As to the question of the use of modern machinery for the construction of engineering work of this kind, conditions in the Orient differ greatly from those in America, where contractors have proper labor-saving machines. In Japan it takes months or even a year to import and learn the use of new machinery.

For mining works, for instance, which are of more permanent character than those for this kind of engineering works, new machines are being introduced from time to time so as to make the workings cheaper and at the same time more rapid.

As to the use of puzzolana in concrete blocks, it may be of interest to mention that, in a volcanic country like Japan, puzzolana is found in many parts of the empire. The writer had opportunities of using this natural cement in several engineering works with which he was connected. The puzzolana from Goto Islands, referred to by the author, is of very good quality. In building the first graving dock in Japan, in the Yokosuka Navy Yard (near the landing place of Commander Perry), puzzolana from the Province of Idzu was used and was satisfactory, especially in the early sixties, when the manufacture of Portland cement was not known in Japan, and imported cement was both very expensive and very rare.

Mr. Jacobs. Charles M. Jacobs, M. Am. Soc. C. E.—The Japanese engineers are to be congratulated on avoiding one of the great errors made in



ENTRANCE OF DOCK NO. 3

NAGASAKI, JAPAN

the early period of graving dock building, namely, in the width of the entrance. They have avoided this, which has detracted from the usefulness of a large number of docks in Europe; but the speaker suggests that they have committed another error in not dividing the dock by a central caisson or by gates. For example, the dock could be subdivided into two equal lengths. Looking at the photograph, Plate VII, it will be observed that a number of torpedo boats are on the blocks, the result being that the repairs to all must be completed, at least temporarily, before floating out. The length of ordinary trading vessels on the coasts of Japan and China averages from 320 to 360 ft. Assuming that a seriously damaged vessel arrives in the dock for repairs, the entire dock is immediately closed for other work until the damaged vessel is ready for refloating. If there had been a midway caisson, or gates, the seriously damaged vessel could remain at the upper end while the outer section of the dock would remain free for docking vessels requiring light repairs, painting, cleansing, etc. Therefore the commercial possibilities and utility of the dock, with comparatively small extra expense, would have been enhanced considerably by its subdivision, and it would still be available when required for ships of the largest tonnage. Mr. Jacobs.

Another point in the paper is surprising, and that is the expense of the adoption of cut stone overlying the concrete, which, according to the figures of costs given in the table, was about seven times the cost of concrete. It is beyond all question that the utilization of concrete alone for graving dock construction has long passed the experimental stage, and is assuming enormous proportions in all classes of engineering structures. The experience of the speaker, in building a short time ago two large graving docks in South Wales entirely of concrete, has proven that concrete is absolutely satisfactory in maintenance, notwithstanding that the walls of this Welsh dock had to pass through most treacherous quicksand before reaching a gravel foundation, and that the total rise and fall of the tides was 32 ft. Therefore it seems to be an unnecessary extravagance to place a granite lining on top of the concrete when the latter is just as efficient, if care be taken to enrich the outer surfaces of the concrete on the sides, altars, steps, and coping.

The speaker hopes he may also be pardoned for criticising the assertion in regard to the non-use of mechanical power, notwithstanding the statement as to the cheapness of labor in Japan. Take the cost of the dock lately completed by the speaker in South Wales, of nearly the same dimensions as the Nagasaki Dock. The contract price for the Welsh dock, built entirely of concrete, and having central gates, was $445 000, whereas the cost of the Japanese dock, notwithstanding the cheap labor, was $700 000. The cost of labor in Great Britain is very much higher than in Japan, and, with the

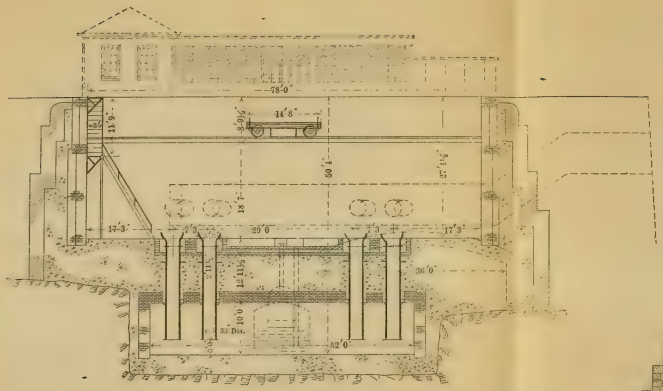
Mr. Jacobs. utilization of mechanical power, the costs have proven more economical.

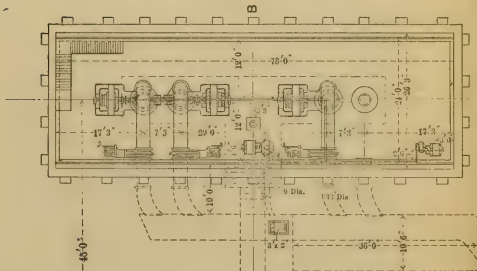
Another point in the paper to which attention has been called is the utilization of puzzolana mixed with the concrete to increase its imperviousness. The great difficulty in all graving dock construction is to obtain water-tightness, due to intermittent stresses on the structure. Particulars of this important feature have been omitted. If the author could give some records of filtration through the walls and invert it would add very much to the interest of the paper.

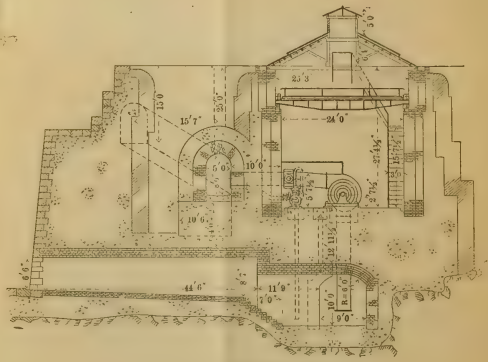
Mr. Hollyday. R. C. HOLLYDAY, M. AM. SOC. C. E.—The speaker, having been engaged in the construction of dry docks for the Navy Department during the past 10 years, believes that it may be of interest to state what has been done.

As Engineer-in-Charge, the speaker was connected with the construction of the timber dry dock at the Puget Sound Naval Station, completed about 1896, at a cost of $700 000. This dock rests on piles, with a concrete foundation under the bottom and for 7 ft. on the sides under the altars. The entrance of the dock, for 75 ft., including, of course, both gate seats, is of concrete faced with cut stone. The pumps are operated by steam engines. It is the only timber dock constructed by the Navy Department which has proved serviceable or at all satisfactory. At about the same time that this dock was being built, a timber dock was being built at the Port Royal Naval Station, South Carolina. The latter has never been satisfactory, and has seldom been used; to-day, it is scarcely in condition to be used at all, and certainly not for any first-class battleship. It is slightly smaller than the Puget Sound dock. At about the same time, Dry Dock No. 3 was being constructed at the New York Navy Yard. This dock is, approximately, of the same size as the Puget Sound dock, being about 625 ft. on the floor and 668 ft. in length over all. It was operated originally by steam-driven pumps, but a new pumping plant has recently been installed for this dock and Dry Dock No. 2, one plant being connected for both, and operated by three motor-driven 45-in. centrifugal pumps, each having an average capacity of 50 000 gal. per min. The original cost of this dock was $794 000. A great deal of trouble was encountered during its construction, and a large part of it was reconstructed almost immediately after it was completed. The cost of reconstruction was about $300 000, and even then the dock was in a very unsatisfactory condition. Three years ago the dock had to be put out of commission for about 6 months during repairs. The repairs were of a rather novel character, and pertained to the foundations near the entrance, principally in cutting off a large stream of water which entered the dock from the harbor. This flow of water undermined the material laid under the dock and under its sides to such an

PUMPING PLANT OF No. 3 DOCK







SECTION ON A-B

extent that it was likely to give way at this point. A great deal of the underlying material was quicksand, which caused most of the trouble in the construction and in its maintenance. It is hoped that this dock may be kept in commission until the completion of Dry Dock No. 4, which is now being constructed at the New York Navy Yard, and then rebuilt and made a masonry dock of concrete, lined with cut stone. Mr. Hollyday.

Dry Dock No. 2 at the Mare Island Navy Yard, California, which the speaker was next connected with, as Engineer-in-Charge, is now under construction. It was originally intended to be of timber, but, by Act of Congress, it was ordered that it be of stone.

The next dock with which the speaker was connected, as Engineer-in-Charge, was one of masonry at the Boston Navy Yard. This was completed during 1905. It is a masonry dock, of concrete faced with granite, and is 788 ft. in length. It was authorized at the same time as the masonry dock for the Portsmouth, New Hampshire, Navy Yard, which, by the way, is located across the river from Portsmouth, at Kittery, Me. This and the Portsmouth dock are approximately the same size; both are operated by electrically-driven pumps, and have all the accessories, in the nature of winches and capstans for handling vessels, all electrically operated. They are also provided with pneumatic pipe lines for furnishing power for work on ships in dock; with conduits for electric wires for power and for lighting; and with water piping, with outlets at frequent intervals, for attaching hose for washing down the dock and for whatever purposes water may be needed there. They are docks of the highest type yet completed, and have all the modern appliances for handling ships expeditiously, economically, and efficiently, and for doing necessary work on ships after they are docked.

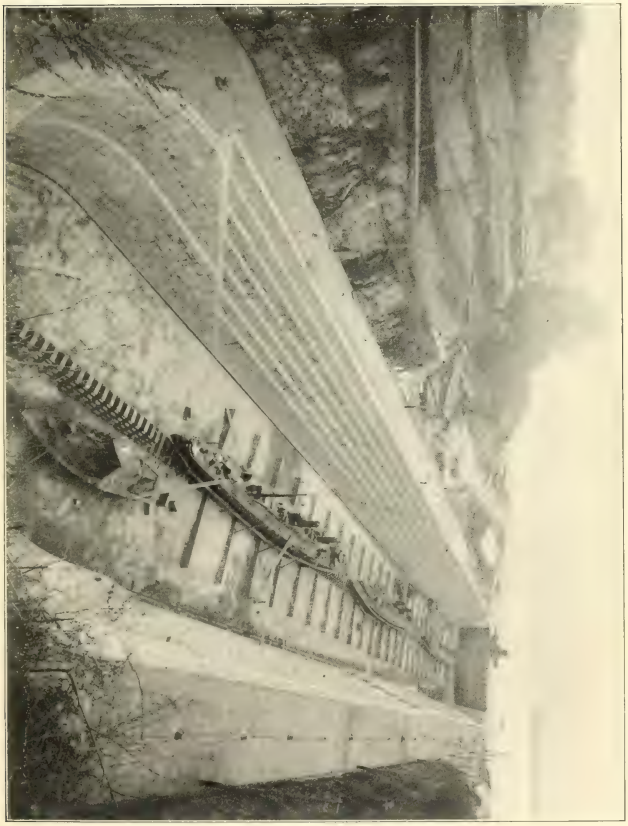
The dock with which the speaker is at present connected, in the capacity of Engineer-in-Charge of construction, is Dry Dock No. 4, at the New York Navy Yard. This is to be of concrete, and little or no cut stone will be used for facing, in fact, none except at the gate seats. The dock is to be 516 ft. long on the floor by 78 ft., 542 ft. on the top by 130 ft., and will have a depth of 31 ft. of water over the sill. The contract for this dock was made during 1905, and 42 months are allowed for completion. It is to be operated by electrically-driven centrifugal pumps. Owing to the limit of cost for this dock, and also the limited space available at the New York Navy Yard, it is not possible to make it as long as the other docks which have recently been authorized by Congress for the Navy Department. It is large enough to dock the largest battleship, and it is not probable that a battleship will ever be built so large as not to be able to enter it. It is also large enough to take in the largest cruisers yet built, but it is not at all improbable that, in the future,

Mr. Hollyday. cruisers may be built for the Navy which will be too large to enter this dock. However, Dry Dock No. 3 is long enough to take in any cruiser which is likely to be built.

Masonry docks are also being built at the navy yards at League Island, Charleston, and Norfolk. All are large docks, and of the highest type, having all the modern appliances of the Portsmouth and Boston docks.

Mr. Jacobs states that probably the Japanese made a mistake in going to the expense of facing the dock at Nagasaki with cut stone. In the speaker's opinion, this was a very important feature, and along the right lines. There are two ways of looking at this question: one from the commercial standpoint, and the other from a Government or engineering standpoint. From a commercial standpoint, it is necessary to design and build a dock which will pay a reasonable interest on the investment. A commercial concern, in building a dock, may not have the capital to invest, and, in any event, cannot afford, from a business standpoint, to make an investment which is known in advance will not yield a reasonable interest. The Government and engineering standpoints are almost the same. In each case it is desirable to build the best structure possible at anything less than what might be called a prohibitive cost. From the Government standpoint, durability and efficiency are the essential features, and to have a structure which will always be available for use whenever it may be needed—a structure where extensive repairs are not continually needed. When repairs are being made it is frequently necessary to put the dock out of commission; at such a time it would not be available for use, and that might be the very time it might be needed most.

At the New York Navy Yard there are three docks, and a fourth one is under construction. Dry Dock No. 1, of masonry, with granite facing; Dry Dock No. 2, originally of timber, completed in 1890, reconstructed and built of masonry with concrete facing and metal strips for protecting the edge of the altar steps and coping, the reconstruction being completed in 1902; Dry Dock No. 3, of timber, completed in 1897. The cost of repairs on the bodies of these docks from January, 1903, to date, has been, approximately: Dry Dock No. 1, nothing; Dry Dock No. 2, $950; Dry Dock No. 3, $17 200. Although Dry Dock No. 2 has only been completed about three years, the concrete walls have begun to give trouble. Water finds its way through the walls at different points, and is first shown by the action of frost during the winter, the cracks become enlarged, the concrete is gradually spawled off the face, and the disintegration goes on. Repairs have been made to the facing, but these repairs are only regarded as temporary, patching work, and it is not believed possible to repair the dock so that there will not be future trouble



GRAVING DOCK NO. 3, OF THE MITSU BISHI COMPANY, AT NAGASAKI, JAPAN.

of the same kind. This trouble would not have occurred had the dock been faced with cut stone, and this illustrates very clearly the point which the speaker makes in regard to facing the dock at Nagasaki with cut stone rather than concrete. Mr. Hollyday.

Originally, the Government built only masonry docks faced with cut stone, and the stone was frequently cut with much greater nicety than was necessary, but this was rather a matter of detail and cost. Dry Dock No. 1 at the Boston Navy Yard is of stone, and was completed in 1833. Dry Dock No. 1 at the Mare Island Navy Yard was completed in 1891; it was built by day labor, and, owing to interruption of appropriations, the work was not continuous and it took some 12 years to build it. Dry Dock No. 1 at the New York Navy Yard is of stone, and was completed about 1846. Dry Dock No. 1 at Norfolk is of stone, and was completed in 1827. These four stone docks—the first docks built by the Navy Department—may be said to be in practically as perfect a state of preservation as ever. During all these years practically no money has been spent on the body of these docks in the way of repairs, whereas, all the timber dry docks built by the Navy Department and completed within the last 10 or 12 years, with the exception of the Puget Sound dry dock, have required very extensive repairs, and one of them is practically useless to-day. This ought to show pretty conclusively that the only course open to the Government is to build the most substantial structure possible.

This Society may not be aware of the fact, but an Informal Discussion on "Dry Docks—Stone *vs.* Wood,"* had considerable to do with influencing the policy of the Government in regard to the kind of docks to be built. In 1898 Congress authorized the construction of four large dry docks—at Portsmouth, Boston, League Island, and Mare Island Navy Yards—the Portsmouth and Boston docks were to be of masonry, and the League Island and Mare Island docks were to be of timber. At that time opinion in the Navy Department was not unanimous as to the kind of dock which should be built, and there was a strong interest favoring the construction of timber docks. Congress had made up its mind to build at least two timber dry docks, against the recommendation of the Secretary of the Navy, as advised by Rear Admiral M. T. Endicott, M. Am. Soc. C. E., Chief of the Bureau of Yards and Docks, and Rear Admiral G. W. Melville, Hon. M. Am. Soc. C. E., Chief of the Bureau of Steam Engineering. There was a great deal of discussion as to the type of dock to be built, but Congress did not change its decision. Under the authority vested in him, the Secretary of the Navy proceeded to have plans prepared, advertise, and make contracts, for the construction of two stone docks at the Portsmouth and Boston

* *Transactions*, Am. Soc. C. E., Vol. XLI, p. 554.

Mr. Hollyday. Navy Yards. He also took the necessary steps for building the two timber docks authorized for the League Island and Mare Island Navy Yards, with a view to having them changed to stone docks by Act of Congress later. During this time, the subject of stone *versus* timber docks was brought up and discussed before this Society, and it was found that those who took part in the discussion strongly favored the stone dry dock. When Congress met, during the following year, members of the Naval Committee were informed that the question of stone *versus* timber dry docks had been discussed very thoroughly before this Society, and that the consensus of opinion of those who had discussed the subject was strongly in favor of stone dry docks; that Congress had made a mistake in adhering to the policy of building timber dry docks, and that it was not too late to rectify the error. The speaker was informed, by one of the most prominent members of the Naval Committee, that the discussion before this Society had as much weight as, or more than, any other one thing in influencing Congress in reversing its policy and authorizing the construction of the League Island and Mare Island docks of masonry.

It may not appear to members of this Society that this is a matter of very much importance, but to the speaker it is, in that this discussion could change the policy of the Government from a wrong principle to a right one.

Although this Society disclaims all responsibility for the facts and opinions advanced in any of its publications, its policy has always been and now is to discuss engineering subjects from an engineering standpoint. It has been against entangling alliances, of all kinds and descriptions. There has never been any suspicion of its furthering the interests of any man or set of men, and by this wise policy it has come about that its opinion is turned to as one of absolute disinterestedness.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The author was especially fortunate in having had available such a good site for a dock. Such favorable locations are very rare. Undoubtedly, the entire base of the dock should rest on solid rock, wherever possible.

The depth of water at the site of the coffer-dam, 50 ft. at high water, to bed-rock, although somewhat excessive, is often exceeded, and special precautions have to be taken on account of the great pressure on the dam when the enclosure is pumped out. A rock-filled dam is generally adopted in such cases, but practice differs with regard to the puddle core.

A semi-circular dam of small stone, properly filled with good clayey material to choke the voids on the outer slope often fulfils all requirements where that slope is not exposed to heavy weather. Where labor is so cheap and the rock excavation is seamy, as in this

case, it is certainly wise to conduct the work with hand labor throughout, as the likelihood of injuring the foundation rock forbids all blasting near the lining work. Mr. Le Conte.

The writer is pleased with the proportions of the concrete, and with the addition of puzzolana to make impervious work, which was very desirable; he is somewhat surprised, however, to note the heavy expense for cutstone facing—$167 500—the average rate being $15 per cu. yd. in place. In these days of economical construction it is not at all clear why the altars, at least, could not have been made of high-grade concrete, if not the entire dock lining.

The writer knows that comparisons are always odious, because conditions are so different in different cases, but thinks that some broad general comparisons would not be out of place.

The new dock at Mare Island, California, now under construction, is on a pile foundation throughout, the dock foundation alone calling for 14 000 piles. The site is quite unfavorable. The dock will be 734 by 120 by 35 ft., and will have an aggregate capacity of 114 180 cu. yd. When completed it will probably cost in the neighborhood of $1 800 000, making a capacity rate of $15.76 per cu. yd.

The Hamilton Graving Dock, at Malta, completed in 1893-94, was founded entirely on rock, but the formation was full of fissures, some of them 8 in. wide, and much water had to be contended with. Besides the cost of the dock proper, there were also included the cost of a factory for repairs, a 160-ton hydraulic crane, and shops and stores for repair and storage of gun mountings, etc. All this brought the cost up to $1 023 163. The dock is 558 by 126 by 44 ft., and has an aggregate capacity of 114 576 cu. yd., which makes the cost $8.93 per cu. yd. Next comes the author's dock, having a cost of $5.56 per cu. yd. of capacity, and finally, the new Hunter's Point Dry Dock, San Francisco, Cal., completed in 1903. This dock is founded throughout on rock. The lining is almost entirely of concrete, excepting the masonry piers at the entrance. The dimensions are 750 by 122 by 36 ft. = 122 000 cu. yd. Of this, 100 000 cu. yd. was rock excavation. The cost of engineering construction alone has been only $488 000, making the cost $4 per cu. yd. The altars on both sides of the dock are entirely of high-grade concrete.

This stands out in strong contrast with the Mare Island Dock, above mentioned, which is situated only a few miles further inland.

CHARLES ALBERTSON, M. AM. SOC. C. E. (by letter).—The original graving dock in Japan was not a graving dock at all, in the present sense of the word. It was simply a sheltered cove or a sandy beach protected by nearby headlands from the severe winds and typhoons which occur at certain seasons. These beaches are found all along the coast lines of the four main, and the many lesser, islands which compose the Island Kingdom. Even to this day, the largest junks Mr. Albertson.

Mr. Albertson. are beached in the old-time way, in order to clean the hulls and make minor repairs during the successive periods between high tides.

For centuries the Japanese had been experienced in the control of water for purposes of irrigation, therefore it is quite probable that the next step in their docking operations was to build a small dam and gate, sufficient at least to protect the little inlets from the peak of the high tide.

From such small beginnings to the great graving dock described by Dr. Shiraishi is a tremendous leap, but the Japanese have shown themselves capable of doing just such things.

One would scarcely dare maintain that the necessity for such docks or the ability to design and build them could have come about except for contact with the nations of Europe and America. Their docks are based on the best foreign practice, but are slightly modified to suit the local demands and conditions. Nevertheless, it is no small matter to build a good dock in Japan, owing to the inexperienced, careless, heedless labor which must be used on such works. The laborer does his work in a very mechanical manner, and all his thinking must be done for him. It is a pleasure to know that Dr. Shiraishi has been entirely successful in the construction of his dock.

Japan is fortunate in having an indented, rather rocky, sloping shore line, which is naturally adapted to dock construction. The Japanese have made good use of these conditions, and most of their dozen or more large private docks are built directly on a rock foundation. They are all faced with native granite. The pump-houses are largely below ground, and contain electrically-driven, direct-connected, centrifugal pumping outfits. Dr. Shiraishi's article describes Japan's present standard practice.

Mr. Jacobs, in discussing the paper, stated that another gate in the middle of the dock would have been an improvement on the design, as one or two small vessels could have then been docked separately. While this might be true, ordinarily, if other docks in the immediate vicinity were not available, and but few large vessels were to be docked, it does not hold in the case of the Nagasaki dock. The owners, The Mitsu Bishi Company, have in operation two other docks and one patent marine railway. The dimensions of these smaller docks are: length on keel-blocks, 510 and 350 ft.; bottom entrance width, 77 and 53 ft.; and depth of water on keel-blocks, $26\frac{1}{2}$ and 24 ft., respectively. The marine railway can accommodate a vessel of 1 000 tons. These, with the other small shipyards in the neighborhood, fully cover the docking requirements of Nagasaki's beautiful harbor.

In regard to cement, the writer shares the opinion which is general among foreigners in Japan, that the best Japanese cement is not equal to the best imported article. Some evidence of this is

given by the Japanese themselves, in that the new dry dock, of the Kawasaki Dock Yard Company, Limited, at Kobe, was built with imported cement. This dock cost more than the big Nagasaki dock, although it is not nearly as large, since it measures 426 ft. from caisson to head wall toe, bottom entrance width 52 ft., and depth of sill 24 ft. The great cost was due to the company's shipbuilding plant having been located on bad ground, without considering dry-dock requirements. This necessitated piling and a large quantity of concrete. The Mitsu Bishi Company profited by the experience of the Kawasaki Company, and have now in use in Kobe Harbor a steel floating dock of 7 000 tons capacity. Mr. Albertson.

Before giving up his residence in Japan, the writer saw the early stages of the construction of the Nagasaki dock. He is of the belief that, under the conditions existing then and there, only a little more machinery could have been used either to advance the work or decrease the cost. The thought is that perhaps an addition to the limited hauling equipment, or even a further installation of some very simple type of conveying apparatus might have been desirable. This, probably, would not have decreased the cost to the contractor, although it should have reduced by a little the time taken in construction; but, then, time is not considered as valuable in Japan as it is in America.

The writer feels that the credit for the design as well as the construction of this the greatest dock in the Far East properly belongs to Dr. Shiraishi, although he modestly refrains from giving even a clue as to his continued connection with the project.

L. F. Bellinger, M. Am. Soc. C. E. (by letter).—Mr. Hollyday states that practically no money has been spent for repairs on the body of Dry Dock No. 1, at the New York Navy Yard; but, as a matter of interest, it is well to note the fact that the entrance to that dock was practically rebuilt some years ago by P. C. Asserson, Civil Engineer, U. S. Navy. On account of quicksand, the old entrance to the dock began to bulge upward. The rebuilding consisted of an inverted granite arch. This work was completed about August 1st, 1888, the total cost being about $86 000. Mr. Bellinger.

The repairs mentioned as being novel by Mr. Hollyday were completed under his direction, therefore, it would be of considerable interest if he would give the Society a detailed account, especially as they pertained to the injection of cement into sand to serve as a cut-off for water percolation, a subject on which information has been requested in the technical papers without eliciting any replies.

Naoji Shiraishi, M. Am. Soc. C. E. (by letter).—In reviewing the discussion, the writer wishes to explain a few points which have been raised. M. Shiraishi.

The advantage of a central partition in the dock is maintained

Mr. Shiraishi. by Mr. Jacobs, and the writer would have no inclination to controvert this suggestion, if the dock were considered independently of surrounding circumstances. In the present case the manager of the shipyard, who has been in the service more than twenty years, did not recognize the necessity for the partition, because of the existence of two other shorter docks and a shipway, which are ample for the accommodation of the number of small steamers calling at the harbor, even after making allowance for any future increase. Mr. Albertson conceives the same idea, and has given the dimensions of the other docks.

The use of puzzolana to increase the imperviousness of concrete was not on account of any preconceived theory. In endeavoring to stop leakages in old docks, puzzolana-concrete was sometimes used rather for economy. The result, in regard to imperviousness, was always found to be better than with ordinary concrete. If puzzolana-concrete had been more expensive than ordinary cement-concrete, the writer would have made laboratory experiments, in order to compare the rates of percolation, before adopting the first; but as the cost of puzzolana is less than half that of cement, he had no hesitation in mixing them. Thus far, the finished work has been free from leakage.

As a scientific investigation, to which others may refer, however, the writer acknowledges the force of Mr. Jacobs' remark that the paper is wanting in records of filtration. Boxes of mortar have since been prepared, some with puzzolana mixed with cement and sand, and some with cement and sand only. Small gas pipes are embedded in the boxes to serve as inlets for pumping in water. By this means, hydraulic pressures will be given to the sides and bottoms of the boxes, and the rates of percolation, for these different kinds of mortar, will be compared. The writer is sorry not to be able to report the results herein, but cannot postpone his concluding remarks until the experiments are finished.

Mr. Jacobs criticizes the assertion regarding the non-use of mechanical power in the construction of this dock. The writer must not be misunderstood as disclaiming the use of machinery, in the same way as cab-drivers protested against locomotives in olden times. For instance, regarding excavation, the writer first had to estimate the prices of rock drills, air compressors, cranes, etc., and then calculate the unit cost of excavation in comparison with what could be done without those machines. He found the latter to be the cheaper. As Mr. Otagawa remarks, the contractors here are not provided with such machines, and their first cost must be included in the excavation estimate. As a reason for his criticism, Mr. Jacobs compares the cost of the dock he constructed in South Wales, $445 000, and that of the Nagasaki dock, $700 000. It is difficult,

however, to ascertain how much of this difference in cost was due to the utilization of mechanical power. The relative cost of docks, given by him, and also by other discussors, is no criterion as to the propriety of the design or the method of execution, local conditions being widely different; and, as Mr. Le Conte remarks, such comparisons are odious. Mr. Shiraishi.

The rapid progress in the use of concrete in various fields of construction, on account of its cheapness and strength, seems to sway the judgment of many in recommending all-concrete for the construction of docks. In the present case, however, the immediate revenue is not the main object, as was explained by the writer at the beginning. The permanent character of the structure and also its appearance were considered worth taking into account. Any repairs or patching up of all-concrete work would distigure the dock badly, although the expense therefor would be of small account. Even in works of utility, of this class, the æsthetic effect commands some consideration.

Another point pertains to a purely local condition; that is, on account of some failures of concrete in submarine works, the writer had reason to doubt the uniform character of Japanese cement. Some barrels might not come up to the standard of the specimens tested, which fact, combined with some possible lack of supervision, might cause patch-work later; whereas, the cost of imported cement is 6 or 7 yens per barrel (4 cu. ft.), and often cannot readily be found on the market.

The writer, however, does not intend to adhere absolutely to stone masonry facing, and is glad to learn of the experience of others regarding the behavior of all-concrete, so that he may break off his conservatism with confidence.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1017.

THE THEORY OF CONTINUOUS COLUMNS.*

By Ernst F. Jonson, Assoc. M. Am. Soc. C. E.

A column which is continuous through two or more stories differs from a one-story column in that its strength in any story is a function, not merely of its dimensions in that story, but of its dimensions in all stories. A 15-ft. section of a continuous column is evidently stronger when the adjoining sections are 10 ft. long than when they are 15 ft. long, other things being equal. Practically all columns used in buildings of more than one story are continuous.

It is the writer's purpose to develop the exact theory of continuous columns in order to deduce from it a simple method of calculating the effect of eccentric loading, both at the floor level and at an intermediate point.

The development of the equation of the elastic curve, though not new, will be given for the sake of convenience and completeness.

The bending moment in a column (Fig. 1) is

$$M = W y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{I}$$

where W is the load, and y the distance from the load line to the axis of the column.

* Presented at the meeting of March 7th, 1906.

The corresponding stress is

$$s = \frac{W a y}{I} \quad \text{.....II}$$

where a is the distance to the extreme fiber, and I is the moment of inertia. The corresponding elongation per unit of length is

$$\epsilon = \frac{W a y}{E I} \quad \text{.....III}$$

where E is the modulus of elasticity.

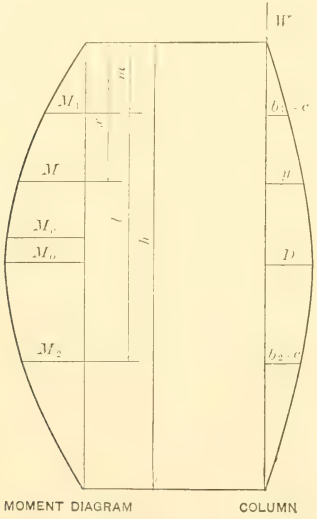


FIG. 1.

The corresponding second differential may, with sufficent accuracy, be taken as

$$\frac{d^2 y}{dx^2} = -\frac{W y}{E I} \quad \text{.....IV}$$

Hence the equation of the elastic curve is

$$y = D \sin. x \sqrt{\frac{W}{E I}} \quad \text{.....V}$$

where $D = y_{max}$.

By multiplying by W we get the equation of the bending moments

$$M = M_o \sin. x \sqrt{\frac{W}{EI}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{VI}$$

where $M_o = M_{max.}$

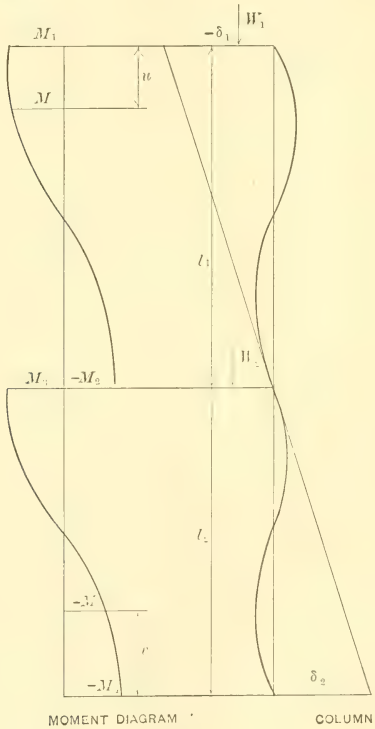


FIG. 2.

The bending of a continuous column in two adjoining stories must fulfil the following condition (Fig. 2):

$$\frac{\delta_1}{l_1} + \frac{\delta_2}{l_2} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{VII}$$

where δ_1 is the deflection of the upper end of the upper column, and δ_2 that of the lower end of the lower one, both measured from the tangent to the elastic curve at the intermediate floor line, and where l_1 is the length of the upper and l_2 that of the lower column.

$$\delta_1 = \frac{1}{E I_1} \int_o^{l_1} u \, M \, d \, u \ldots\ldots\ldots\ldots\ldots\ldots \text{VIII}$$

$$\delta_2 = \frac{1}{E I_2} \int_o^{l_2} v \, M \, d \, v \ldots\ldots\ldots\ldots\ldots\ldots \text{IX}$$

where I_1 is the moment of inertia of the upper, and I_2 that of the lower column, and where u is the distance from the top of the upper, and v that from the bottom of the lower column.

Hence Equation VII becomes:

$$\frac{1}{I_1 \, l_1} \int_o^{l_1} u \, M \, d \, u + \frac{1}{I_2 \, l_2} \int_o^{l_2} v \, M \, d \, v = 0 \ldots\ldots\ldots \text{X}$$

Let $x - u = m$,

$$\int_o^l u \, M \, d \, u = M_o \int_o^{m+l} x \sin. \, x \sqrt{\frac{W}{E I_1}} \, d \, x$$

$$- M_o \int_o^m x \sin. \, x \sqrt{\frac{W}{E I_1}} \, d \, x - M_o \, m \int_o^{m+l} \sin. \, x \sqrt{\frac{W}{E I}} \, d \, x$$

$$+ M_o \, m \int_o^m \sin. \, x \sqrt{\frac{W}{E I}} \, d \, x,$$

hence

$$\int_o^l u \, M \, d \, u = M_o \frac{E I}{W} \left[\sin. \, (m+l) \sqrt{\frac{W}{E I}} \right.$$

$$\left. - (m+l) \sqrt{\frac{W}{E I}} \cos. \, (m+l) \sqrt{\frac{W}{E I}} \right]$$

$$- M_o \frac{E I}{W} \left[\sin. \, m \sqrt{\frac{W}{E I}} - m \sqrt{\frac{W}{E I}} \cos. \, m \sqrt{\frac{W}{E I}} \right]$$

$$- M_o \, m \sqrt{\frac{E I}{W}} \left[1 - \cos. \, (m+l) \sqrt{\frac{W}{E I}} \right]$$

$$+ M_o \, m \sqrt{\frac{E I}{W}} \left[1 - \cos. \, m \sqrt{\frac{W}{E I}} \right].$$

$$\int_o^l u \, M \, d \, u = M_o \frac{E I}{W} \left[\sin. \, (m+l) \sqrt{\frac{W}{E I}} \right.$$

$$\left. - l \sqrt{\frac{W}{E I}} \cos. \, (m+l) \sqrt{\frac{W}{E I}} - \sin. \, m \sqrt{\frac{W}{E I}} \right] \ldots\ldots \text{XI}$$

From the Moment Equation VI, we find

$$M_1 = M_o \sin. m \sqrt{\frac{W}{EI}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{XII}$$

$$M_2 = M_o \sin. (m+l) \sqrt{\frac{W}{EI}} \quad \ldots\ldots\ldots\ldots\ldots \text{XIII}$$

$$\sin. m \sqrt{\frac{W}{EI}} = \sin. (m+l) \sqrt{\frac{W}{EI}} \cos. l \sqrt{\frac{W}{EI}}$$

$$- \cos. (m+l) \sqrt{\frac{W}{EI}} \sin. l \sqrt{\frac{W}{EI}}.$$

$$\frac{\sin. (m+l) \sqrt{\frac{W}{EI}} \cos. l \sqrt{\frac{W}{EI}} - \sin. m \sqrt{\frac{W}{EI}}}{\sin. l \sqrt{\frac{W}{EI}}} = \cos.(m+l) \sqrt{\frac{W}{EI}}.$$

$$\frac{M_2 \cos. l \sqrt{\frac{W}{EI}} - M_1}{\sin. l \sqrt{\frac{W}{EI}}} = M_o \cos. (m+l) \sqrt{\frac{W}{EI}} \quad \ldots\ldots \text{XIV}$$

Introducing these values into Equation XI, we get:

$$\int_o^l u M du = \frac{EI}{W} \left(M_2 - l \sqrt{\frac{W}{EI}} \frac{M_2 \cos. l \sqrt{\frac{W}{EI}} - M_1}{\sin. l \sqrt{\frac{W}{EI}}} - M_1 \right)$$

$$\int_o^l u M du = \frac{EI}{W} \left[M_1 \left(\frac{l \sqrt{\frac{W}{EI}}}{\sin. l \sqrt{\frac{W}{EI}}} - 1 \right) \right.$$

$$\left. + M_2 \left(1 - \frac{l}{\tan. l \sqrt{\frac{W}{EI}}} \sqrt{\frac{W}{EI}} \right) \right] \quad \ldots\ldots\ldots\ldots \text{XV}$$

Introducing this value into Equation X, we get:

$$\frac{M_1}{W_1 l_1} \left(\frac{l_1 \sqrt{\frac{W_1}{E I_1}}}{\sin. l_1 \sqrt{\frac{W_1}{E I_1}}} - 1 \right) + \frac{M_2}{W_1 l_1} \left(1 - \frac{l_1 \sqrt{\frac{W_1}{E I_1}}}{\tan. l_1 \sqrt{\frac{W_1}{E I_1}}} \right)$$

$$+\frac{M_3}{W_2 l_2}\left(1-\frac{l_2\sqrt{\frac{W_2}{E I_2}}}{\tan.\ l_2\sqrt{\frac{W_2}{E I_2}}}\right)$$

$$+\frac{M_4}{W_2 l_2}\left(\frac{l_2\sqrt{\frac{W_2}{E I_2}}}{\sin.\ l_2\sqrt{\frac{W_2}{E I_2}}}-1\right)=0\ldots\ldots\text{XVI}$$

This equation is a "theorem of four moments" for continuous columns.

We also know that

$$M_2-M_3+M_e=0\ldots\ldots\ldots\ldots\ldots\ldots\text{XVII}$$

where M_e is the moment due to eccentricity of loading, the left-hand side being the positive side.

By applying these two equations to each story, from top to bottom of a continuous column, the two end moments of each section of the column are found in terms of the lower end moment of the column below, and by working back from bottom to top, introducing the values of the lower end moment, the absolute values will be found.

From Equations XII and XIII we find the maximum moment

$$M_o=\frac{\sqrt{M^2{}_1+M^2{}_2-2M_1M_2\cos.\ l\sqrt{\frac{W}{E I}}}}{\sin.\ l\sqrt{\frac{W}{E I}}}\ldots\ldots\text{XVIII}$$

From Equation VI we get the distance, h, between two points in which the elastic curve, if prolonged, would intersect the load line.

$$h=\pi\sqrt{\frac{E I}{W}}\ldots\ldots\ldots\ldots\ldots\ldots\text{XIX}$$

This is Euler's formula for the length of an ideal, centrally-loaded column. The distances, m and $m+l$, from the end of the ideal column to the two ends of the actual column will be, according to Equation VI,

$$m=\frac{h}{\pi}\text{ arc sin. }\frac{M_1}{M}\ldots\ldots\ldots\ldots\text{XX}$$

$$m+l=\frac{h}{\pi}\text{ arc sin. }\frac{M_2}{M_o}\ldots\ldots\ldots\ldots\text{XXI}$$

From these values of m and $m+l$ it will be seen whether or not the maximum moment falls within the actual length of the column.

Let M_c be the moment at the middle of the column. According to Equation VI

$$M_c = M_o \sin. \left(m + \frac{l}{2}\right) \sqrt{\frac{W}{E\,I}}$$

$$M_c = M_o \left(\sin. m \sqrt{\frac{W}{E\,I}} \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}} + \cos. m \sqrt{\frac{W}{E\,I}} \sin. \frac{l}{2} \sqrt{\frac{W}{E\,I}}\right) \ldots\ldots \text{XXII}$$

According to Equation XIII

$$M_2 = M_o \left(\sin. m \sqrt{\frac{W}{E\,I}} \cos. l \sqrt{\frac{W}{E\,I}} + \cos. m \sqrt{\frac{W}{E\,I}} \sin. l \sqrt{\frac{W}{E\,I}}\right) =$$

$$M_o \left(\sin. m \sqrt{\frac{W}{E\,I}} \cos.^2 \frac{l}{2} \sqrt{\frac{W}{E\,I}} - \sin. m \sqrt{\frac{W}{E\,I}} \sin.^2 \frac{l}{2} \sqrt{\frac{W}{E\,I}} + 2 \cos. m \sqrt{\frac{W}{E\,I}} \sin. \frac{l}{2} \sqrt{\frac{W}{E\,I}} \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}\right)$$

hence

$$M_o \cos. m \sqrt{\frac{W}{E\,I}} \sin. \frac{l}{2} \sqrt{\frac{W}{E\,I}} = \frac{M_2}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}}$$

$$- M_c \frac{\sin. m \sqrt{\frac{W}{E\,I}} \cos.^2 \frac{l}{2} \sqrt{\frac{W}{E\,I}}}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}} + M_o \frac{\sin. m \sqrt{\frac{W}{E\,I}} \sin.^2 \frac{l}{2} \sqrt{\frac{W}{E\,I}}}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}}$$

Inserting this value in Equation XXIII, we have

$$M_c = \frac{M_2}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}} + M_o \frac{1}{2} \sin. m \sqrt{\frac{W}{E\,I}} \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}} + M_o \frac{\sin. m \sqrt{\frac{W}{E\,I}} \sin.^2 \frac{l}{2} \sqrt{\frac{W}{E\,I}}}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}}$$

$$M_c = \frac{M_2 + M_o \sin. m \sqrt{\frac{W}{E\,I}}}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}}$$

Substituting M_1 for $M_o \sin. m \sqrt{\frac{W}{E\,I}}$, as per Equation XII, we have

$$M_c = \frac{M_1 + M_2}{2 \cos. \frac{l}{2}\sqrt{\frac{W}{E\,I}}} \quad \ldots\ldots\ldots\ldots\ldots \text{XXIV}$$

Hence it is seen that when, as is usually the case in buildings, there is but little variation in the function, $\frac{l}{2}\sqrt{\frac{W}{E\,I}}$, for any two adjoining stories, M_c may be taken as proportional to the average of M_1 and M_2. This implies that we may, without great error, neglect the curvature of the moment diagram when the theorem of four moments becomes:*

$$\frac{l_1}{I_1}(M_1 + 2\,M_2) + \frac{l_2}{I_2}(2\,M_3 + M_4) = 0 \ldots\ldots\ldots \text{XXV}$$

Any possible irregularity in a column, such as a bend, an unsymmetrical distribution of the material, or a variation in the modulus of elasticity, may be reduced to an equivalent eccentricity of loading. This equivalent eccentricity, which we will call c, may be deduced from experiments by means of Equation V, if we substitute c for y, $\cos. \frac{l}{2}\sqrt{\frac{W}{E\,I}}$ for $\sin. x \sqrt{\frac{W}{E\,I}}$ (provided that the specimen is supplied with round or knife-edge bearings, as it should be, in order to give reliable results), and insert the actual value of the deflection, which is

$$D = \frac{I\,(s - w)}{a\,W} \quad \ldots\ldots\ldots\ldots\ldots \text{XXVI}$$

hence

$$c = \frac{I\,(s - w)}{a\,W} \cos. \frac{l}{2}\sqrt{\frac{W}{E\,I}}$$

or

$$c = \frac{r^2\,(s - w)}{a\,w} \cos. \frac{l}{2\,r}\sqrt{\frac{w}{E}} \quad \ldots\ldots\ldots \text{XXVII}$$

where r is the radius of gyration, and w the average unit stress.

For immediate practical use, c may be deduced from the particular building law or specification under which the work is to be executed. Thus the writer gets, from the New York building law, a c for steel columns equal to $\frac{1}{16}$ in. per foot of column length, and

*See the writer's paper "The Theory of Frameworks with Rectangular Panels, and its Application to Buildings which have to Resist Wind," *Transactions*, Am. Soc. C. E., Vol. LV, p. 413.

for cast-iron columns it becomes one-third of the radius of gyration. These values are round figures, which, in the writer's opinion, harmonize with the spirit of the law.

The actual values vary with length, radius of gyration, and distance to extreme fiber. This value for c must be added to every actual eccentricity, so that Equation XVII becomes

$$M_2 - M_3 + M_e + W_1 c_1 + W_2 c_2 = 0 \ldots\ldots\ldots \text{XXVIII}$$

When there is considerable difference between the numerical values of M_1 and M_2, M_0 must be calculated by Equation XIX, but when they are about equal and of the same sign the simpler formula, Equation XXIV, may be used, as there will then be but little difference between M_o and M_c. When M_1 and M_2 are of about the same numerical value, but of opposite sign, as is usually the case in buildings, M_o falls outside the actual length of the column and, therefore, need not be considered, the length being seldom greater than half the distance between the maximum moments. The New York building law allows a maximum length of 64% of that distance.

From Equation XXV it is seen that when there is not much difference in the functions, $\frac{l}{I}$ and M, of the various stories, we may assume

$$M_1 = -M_2 = M_3 = -M_4$$

We then get the following simple formula which will cover most cases of eccentric loading occurring in buildings:

$$-M_2 = M_3 = W\left(c + \frac{b}{2}\right) \ldots\ldots\ldots\ldots \text{XXIX}$$

where b is the eccentricity of the resultant load. The only condition to be fulfilled by each of these moments being

$$M_e \overset{=}{<} \frac{I}{a}(p - w) \ldots\ldots\ldots\ldots\ldots\ldots \text{XXX}$$

p being the safe compressive stress of the material.

When there are moments in two directions at right angles to each other, the best way is to calculate the column in the two directions separately, and then add the resulting unit stresses. Equation XXX then becomes

$$\frac{M_{ex}\, a_x}{I_x} + \frac{M_{ey}\, a_y}{I_y} \overset{=}{<} p - w \ldots\ldots\ldots\ldots\ldots \text{XXXI}$$

If an eccentric load be applied to a column at a point inter-

mediate between two fixed points or floor levels, Equation VII becomes (Fig. 3)

$$\frac{\delta_1}{l_1} + \frac{\delta_2}{l_2} = \frac{\alpha_1 - \alpha_2}{l_1} + \frac{\alpha_3 - \alpha_2}{l_2} \quad \text{XXXII}$$

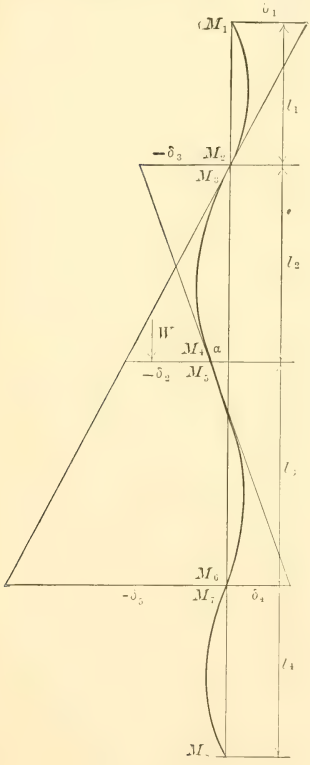


FIG. 3.

where α_1, α_2 and α_3 are the deflections of three successive points at which external forces are applied. We also need an equation covering three sections or column lengths

$$\frac{\delta_1}{l_1} + \frac{\delta_3}{l_2 + l_3} = 0$$

where δ_5 is the deflection of the lower end of the lower section measured from the tangent to the elastic curve at the upper end of the middle section. Hence

$$\delta_5 = \frac{\delta_2 (l_2 + l_3)}{l_2} + \frac{\delta_3 l_3}{l_2} + \delta_4$$

where δ_3 and δ_4 correspond in the middle and bottom sections to δ_1 and δ_2 of the top and middle sections. Hence

$$\frac{\delta_1}{l_1} + \frac{\delta_2}{l_2} + \frac{\delta_3 l_3}{l_2 (l_2 + l_3)} + \frac{\delta_4}{l_2 + l_3} = 0 \ldots\ldots \text{XXXIII}$$

If we now insert the various values of δ in Equations XXXII and XXXIII, we get the final equations. The exact values are given by Equations VIII, IX and XV, but, for practical purposes, the approximate value used in Equation XXV will be sufficiently accurate. The equations then become:

$$\frac{l_1}{I_1} (M_1 + 2 M_2) + \frac{l_2}{I_2} (2 M_3 + M_4) = 6 E \left(\frac{\alpha_1 - \alpha_2}{l_1} + \frac{\alpha_3 - \alpha_2}{l_2} \right) \ldots\ldots \text{XXXIV}$$

$$\frac{l_1}{I_1} (M_1 + 2 M_2) + \frac{l_2}{I_2} (2 M_3 + M_4) + \frac{l_3}{I_2 (l_2 + l_3)} (M_3 + 2 M_4) + \frac{l_3}{I_3 (l_2 + l_3)} (2 M_5 + M_6) = 0 \ldots\ldots \text{XXXV}$$

By means of Equations XXVI and XXXIV we find the various moments, as previously explained; not the absolute value, however, but a value expressed in terms of α, which is the deflection of the unsupported point. The value of α is found by applying Equation XXXV to the two sections above the unsupported point and the one below it. The equation may also be applied to the two sections below and the one above, if it be reversed, that is, if M_1 and M_6, M_2 and M_5, M_3 and M_4, l_1 and l_3, exchange places.

If the column is only two sections long, one above and one below the unsupported point, and pin-connected at both ends, the above formulas cannot be used, for the problem is then a statically determined one. When the bottom end of a column is fixed, the foundation may be considered as a section of the column, the moment of inertia of which is infinite.

As to centrally-loaded columns, the writer would suggest the

following modification of the present practice, namely, that, instead of proportioning a column according to its own length alone, it would be better to take the length as one-half the distance from the middle of the story below to that of the story above. By the present practice the long columns are made relatively stronger than the short ones.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1018.

THE POSITION OF THE CONSTRUCTING ENGINEER, AND HIS DUTIES IN RELATION TO INSPECTION AND THE ENFORCEMENT OF CONTRACTS.*

By Albert J. Himes, M. Am. Soc. C. E.

With Discussion by Messrs. James Smith Haring, W. D. Lovell, Benjamin Thompson, S. Bent Russell, Willard Beahan, W. A. Aiken, Augustus Smith, G. S. Bixby and Albert J. Himes.

In a broad sense, every man who has charge of engineering work is an inspector. His business is to see that the work is executed in a proper manner. The term, inspector, is generally used in a more restricted sense, as meaning a man of more limited power, one whose duties pertain to some one material or class of construction. This paper will deal with inspection in both the broad and the restricted sense, its purpose being to present a comprehensive but general view of the responsibilities and duties of an engineer in charge of construction.

Inspection is subsequent to the execution of a contract, and has to do with its performance. Therefore it presupposes a knowledge of the terms of the contract and of the rights of the parties thereto.

* Presented at the meeting of January 3d, 1906.

A contract is a legal instrument, and the rights of the parties are matters of law. Laws are of two kinds, statutory and common; all having their origin in the old English common law, the basis of which was justice and equality between men. The common or unwritten law, *lex non scripta,* as Blackstone calls it, is founded upon the immemorial customs of the English people, and the statutory or written law, *lex scripta,* is based upon the enactments of legislative bodies, made for the purpose of declaring or remedying the former. It must not be supposed that the common law remains unrecorded, for, since men first learned to write, the customs of the people, as expressed in the decisions of the various courts, have been compiled and preserved for later use. The distinction is that the common law is based upon custom, and the statutory law upon legislation. What follows is stated from the common-law basis, and, though various modifications are to be found among the statutes of the several States, it remains substantially correct as a guide for the enforcement of engineering contracts.

A contract is an agreement between certain parties under which at least one of those parties undertakes to perform a certain act or acts, or to refrain from some specific act or acts. In order that it may be effective as a legal instrument, certain conditions must be fulfilled. First, there must be an offer and acceptance, and in that offer and acceptance is implied a meeting of minds of the parties. By the meeting of minds is meant a clear and definite understanding, by both parties, of the thing or things to be done. Nothing can be more reasonable or logical than this requirement that the parties to a contract shall understand its terms.

The failure to understand, in the beginning, the subject of a contract, is a fruitful source of dissatisfaction. In engineering work there is too much laxity in the preparation of specifications. The specifications of the engineer are always made a part of the contract, and many times they are quite obscure and indefinite. In this is seen a need for greater facility of expression among engineers. They should spend more time, during education, in learning how to use the English language. The technical education is not enough. An engineer should talk and write fluently.

There is also among engineers a lack of appreciation of the necessity of definiteness in their specifications. It often happens

that the engineer who, by reason of experience and ability, is qualified to write the specifications, is so burdened with administrative details that he has no time to spare, and the work is turned over to an office assistant who has no qualification for the work.

This laxity has been a frequent source of trouble on public work where the engineers, coming from work not surrounded by governmental restrictions, have learned by hard experience that both contractors and taxpayers have a right to a strict and impartial interpretation of their contracts, and that, unless clear and specific, such an interpretation is very difficult.

A contract to assume a risk, as an insurance against fire, or an agreement to excavate a channel in the earth, regardless of the character of the material, is binding; but a contractor is not bound to assume a risk not contemplated in the agreement. For instance, in a contract for excavation where different materials are to be paid for at different prices, as clay, gravel, and quicksand, if rock be unexpectedly encountered, no price having been made therefor, the contractor is not bound to remove it. The risk to be assumed must be definitely described in the contract.

The offer and acceptance are usually expressed in writing, and constitute the instrument. Necessarily, they contain all data required for a clear understanding of the subject. Where drawings or samples are needed, to make the subject plain, they may be referred to and described as a part of the instrument. It is only necessary to state what drawings and what samples are meant, and to preserve those drawings and samples so that they may always be clearly identified as the ones referred to.

The second requirement for the legality of a contract is that there shall be a consideration. It would seem to require no law to make it clear that without a consideration there can be no contract. Any other condition would be slavery. One party, clearly, cannot exact something from another, even by agreement, unless some valuable consideration is given in return. But the courts will not inquire into the sufficiency of the consideration unless there be an allegation of fraud. A consideration of one dollar is as good as anything else to establish the legality of a contract, as far as this requirement is concerned.

The common law yields to the statute, where that has effect, and

the latter in turn is restricted by the Constitution. The Federal Constitution provides for freedom among citizens in making contracts, and to question the sufficiency of the consideration for a contract would be an interference with this constitutional liberty.

The third requirement of legality is that the parties to the contract shall be competent. It is clear enough that minors and lunatics, or persons of unsound mind, cannot enter into binding contracts. The statutes provide other restrictions on the ability of certain individuals to enter into a contract.

The fourth requirement is that the object of the contract shall not be contrary to law. A contract to blow up the State capitol would be clearly illegal, as would any other contract to perform an unlawful act.

The fifth requirement is that there shall be some legal evidence of the contract. For this reason it is well to have some knowledge of the rules of evidence. A signature to an agreement is generally sufficient to prove the contract; sometimes a witness is required, and sometimes a seal. In some States a verbal contract is legal, but it is clear that some evidence would be needed to prove that such a contract had been made.

These are the principles of a legal contract. In a specific case it is always necessary to know that the statutes of a State have been complied with, but these principles must always be followed. A failure to do so would render the contract either wholly or partially void.

Having these things in mind, the engineer will take up the papers, plans, and specifications for a contract, and consider its performance. Since this paper is devoted to inspection, nothing will be said about forms of contracts, their virtues and mistakes, or about plans and specifications, except as they bear directly upon the subject.

The first difficulty to be encountered is usually some error or omission in the plans or specifications. This defect must be corrected. The contract, in so far as this item is concerned, is void, for, clearly, there has not been a "meeting of minds" concerning it. If the matter is of minor importance, the contract usually provides that the engineer shall decide what is to be done; but he cannot decide that the matter is one of minor importance. His direction

concerning it assumes such to be the case, and, in following that direction, the contractor accepts the engineer's assumption, but the degree of importance of the matter is a question of fact that may be submitted to a jury on the application of either party to the contract.

Next, there may be found something that is impossible of construction or needlessly expensive, and the contractor may object to it or propose some other method. Again, the same rule applies. The engineer may decide, and the contractor may accept the decision, but either party to the contract may invoke the law, to determine whether the degree of importance of the matter will suffice to annul the contract and render a new agreement necessary. For this reason many contracts provide for supplementary agreements to cover such matters, but, whether or not such agreements have been provided for, they may be made at the option of the parties interested.

These matters have a bearing on contracts made with security for their performance, as a bond or deposit of money. In such cases the errors might afford ground on which to annul the contract and release the security. Such contracts are usually made where one of the parties is a public corporation, and the case affords one example of the necessity of great care in planning public work.

In much of the work of an engineer it is difficult to know in advance just what his client may want before a contract for construction is completed. The president of a railroad may suddenly order a suspension of all operations, or he may make a change of plan that will largely increase or decrease the work to be done. In a government contract, the interested public may pay no attention to a proposed improvement until after the contract is let and partially performed. Then, upon a discovery that something is not to its liking, it will become very active, and use all its political resources to effect a change.

For such reasons as these, it has become common to insert in the contract a clause providing for increase or decrease of work, changes of plan, and delays. Such a clause may or may not be binding, according to how it is used. A contingency is a proper subject for contract, but that contingency must be clearly expressed. No blanket clause will cover contingencies not considered when making the contract.

A blanket clause is often construed liberally in the interest of

the party of the first part, and the interpretation is a source of great dissatisfaction on the part of the contractor. That he does not more often seek redress in the courts is because of the expense and delay attendant thereon. The sum involved must be large, to warrant such a procedure. The theory of the law is held by its devotees to be the discouragement of litigation, and, in this respect, because of the expense and the numerous difficulties and delays in getting a final decision, the legal profession has attained a degree of perfection which engineers may not hope to equal.

In a set of specifications for a large grain elevator, the engineer, in order to make assurance doubly sure, inserted the following:

"If there is anything omitted from these specifications that is necessary to make the elevator and power plant complete and in perfect working order, it will be understood to be herein contained."

The elevator was built, and it would be very interesting to learn whether this clause had any application, and, if so, what. Such a blanket clause is often used with little regard for its application. If the elevator was let for a lump sum, and without detailed specifications, the clause would seem to be sufficiently clear. If there were detailed plans and specifications, the clause would offer an excellent chance for a dispute, which would probably be settled by the contractor yielding, as a matter of policy and with no regard to justice. It is certain that the clause would add nothing to the definiteness of the contract. On the contrary, it would show that the engineer, himself, was somewhat hazy about the matter.

Some contractors exhibit an unusual willingness to carry out the most erratic directions of the engineer. This is done in order to break away from the contract and set up a claim that, by reason of change of plan, the contract has been annulled and a new basis of compensation is necessary. That is chicanery, to be sure, but the engineer should be able to recognize it. Claims against public corporations have many times been treated with excessive liberality, and they should not be allowed to succeed without the engineer's approval.

Suppose the contractor fails to complete his work, or to perform some portion of it in a satisfactory manner. The courts will not enforce a contract. The best that can be done is to secure money damages for his failure. This involves litigation and delay, and, to

avoid these difficulties, it is usual to retain a part of the money due the contractor, and provide that it may be used to complete any portion of the work left unfinished by him.

Being familiar with all conditions surrounding the contract, the inspector goes on the work intending to secure the best results possible, both by lending his aid in every way, and by insisting on the correct performance of the work. He is soon confronted with the problem of deciding whether certain work is good enough to accept or bad enough to reject. It has been said that work or material is either good or bad, that there is no middle ground, that the competent inspector must be able to distinguish clearly the difference, and require only that which is just, never accepting anything at fault and never rejecting that which is good. That is a sound position, if taken generally, not rigidly. Rigidly, it is the dictum of conceit and arrogance; conceit, because it assumes perfection on the part of the inspector, and arrogance, because, born of ignorance, it permits no question to be raised. Absolutism has no place in business. Business men are accustomed to dealing with practical affairs, and have little use for perfection. It is for this reason that a skilled and conscientious engineer may at times hear his efforts referred to contemptuously as "hair-splitting niceties," and find himself classed among men who have no judgment.

All things are relative, and are governed by conditions. First-class ashlar masonry is seldom used for retaining walls, and machine-shop methods are not expected in structural work. If the specifications seem to mean otherwise, they must be very clear and definite, in order to stand. Where the specifications are obscure, or where they conflict, custom will govern; so, if a man has some unusual and particular matter which he wishes observed, in the execution of his plans, he must make it very plain in the contract.

Where a contract specified a rip-rap slope wall, laid by hand, no stone to be less than 1½ cu. ft. in volume, and the engineer placed an assistant on the work to measure every stone that went in the wall, his superior decided, on the appeal of the contractor, that in a rip-rap wall it was not necessary to dress the beds of the stone, and that fillers and small stone might be used, to a limited extent, to fill up the crevices. Walls formerly built in the same vicinity and for the same parties had been built of rough stone levelled up with

spawls, and the engineer was clearly attempting to enforce an unusual and strained interpretation of the specification. The specification was at fault in containing the conflict, but the engineer showed a lack of ordinary good sense in his attempt to decide what was intended.

Most specifications call for first-class work. First-class work is simply the best of current practice, and, therefore, it behooves the engineer or inspector to be as thoroughly posted in current practice as in his own specifications.

Again, the personal equation of the inspector must be recognized. It is an old trick for a contractor to confront a young inspector with, say, a cross-tie which he has rejected and one which he has accepted, and ask him which is which. Every experienced inspector knows that he will at times accept, and at times reject, things which are, so to speak, on the dividing line. He must have an ideal, a standard, and if, through good nature, he accepts things which are not quite up to the standard, he does not escape the difficulty; he simply lowers the standard. If the work could be graded by percentages, and the rule was to accept all that ranked 90%, he would find himself accepting 89% and rejecting 91 per cent. That is the imperfection of human nature, and it cannot be helped. It would not make matters better to drop the standard to 89 per cent. The same conditions would exist. Between narrow limits, the inspector should have full authority to decide what he will take and what he will reject. It is seldom that a contractor objects to the classification of an experienced inspector.

There is a great variety of inspection, and it is performed in many different ways. In any specific case its purpose should be clearly in mind, and the means used should be adapted thereto. It may be desired to examine minutely for the detection of flaws each individual item, as in the case of steel eye-bars for bridges. Or it may be desired merely to exercise a restraining influence by occasional and partial inspection. The latter aim is the more common, and generally results in superficial work of very little value.

These methods may be illustrated by the inspection of railroad cross-ties. Many roads secure considerable quantities of ties from the timber along their own lines. The ties are piled along the right

of way and inspected at leisure. Every tie is handled, and each of its six surfaces is carefully examined. It is then rated as first, second, or cull, and carefully marked. Such rigidity of inspection is not possible where large quantities of ties are purchased and inspected at a place remote from the road. The other extreme is exemplified in the purchase of ties by ship load, the inspection to be made while loading, and the loading done by machinery at the rate of several thousand per hour. In such a case it is possible, by employing a sufficient number of men, to sort out and reject a great many culls, but the work is only partially done, and the best work of the inspector is to form a clear idea as to whether the ties are running good or bad, whether the purpose of the seller is to furnish good ties or to run in as many culls as possible, and then to govern, either the acceptance of the cargo or future purchases, by what he has learned.

This is a case where the contract rights should be definite and well understood, for should the impression of the inspector be unfavorable, there should be some means of refusing the cargo unless more carefully sorted.

Inspection by sampling, as in the case of cement, is another class of work, and there is room for the exercise of judgment in selecting the samples. There should be a very definite understanding as to what portion of a shipment should be accepted or rejected on the result of certain tests. The tests to be applied to cement are a matter of prime importance, and have been the subject of much research and experiment.

If a contractor persists in using cement which has been condemned, and is not restrained from completing the work, but is denied payment, he may take the matter into the courts and endeavor to collect compensation. In that event it would not be enough for the engineer to show that the cement was not in accordance with the specifications. Granting that was established, in order to claim damages, he would be obliged to demonstrate the bad quality of the cement and its resultant injury to the work. That is the supreme test of the specifications, and one which they frequently will not stand. When a man specifies something as a matter of judgment, the necessity for which cannot be readily demonstrated, and the

contractor tries to use something else, the best way is to stop him at once. In doing so, the engineer may bring upon himself the charge of delaying the work, but he must be prepared to meet it.

Controversies of this sort have much to do with the objections to letting work publicly to the lowest bidder. When a man has shown a lack of disposition to carry out work as planned, without a dispute, engineers do not care to have him secure their work. In government work, as in private affairs, if the lowest bidder is irresponsible, his bid may be rejected, but it may be necessary to show his lack of responsibility. To do this without raising a suspicion of fraud requires evidence acceptable to the court, and it may be hard to secure. In private work an opinion is often sufficient.

The inspection of masonry is performed sometimes by men who remain constantly on hand while the masons are at work, and sometimes by members of an engineer corps who go from one piece of work to another, often covering 20 miles of line. To one familiar with masonry construction it is very clear that the latter method is little better than no inspection at all, for large volumes of masonry can be thrown together in the most haphazard way, and with the defects so hidden that only a thorough investigation will find them. Such shiftless inspection is to blame for much of the present disfavor of stone masonry on railroads. Many roads have had large quantities of masonry go to pieces because they were not well built. The fault is not inherent in stone masonry, but in the system of supervising its construction. Stone masonry, well built, should not be less durable than concrete masonry.

The inspection of structural steel at a rolling mill is a special line of work. It is supposed to consist of physical tests of samples cut from the material, and of a careful surface inspection of each bar or plate. At present there is practically no surface inspection, save by the mill employees. The output of the mills is so large that there is no time to handle the material for a foreign inspector. In cases where foreign inspectors are employed for this work, they see so little of the material that it would be far better to throw the whole responsibility on the mills, and make no surface inspection at all. There is a better chance to see the material at the bridge shops, and it may be rejected there if found defective.

The physical tests are made on samples supposed to be cut from

the material ordered, and to represent certain melts in the open-hearth furnace. The foreign inspector does not usually see the specimen cut, and cannot check up the marking of the melt number. The information which he secures he must assume to be correct. His work is no check on the honesty of the manufacturer. He merely ascertains the quality of the material, on the assumption that all information given is correct; but, in the conduct of a steel mill, the records are so numerous and voluminous that falsification is not easy, and any lack of system or any deception is quite likely to be discovered. The real worth of the inspector depends upon his familiarity with the methods of manufacture and his ability to judge of the care and system in use. In this way he can decide quite intelligently whether or not the plant is run properly, and, if dissatisfied, he can ask for evidence that things are as they should be, or advise that material be secured from another plant.

An experience at a large rail mill will illustrate the character and results of one class of inspection: There were several thousand tons of rails to inspect at a plant where the writer had no previous acquaintance. After a brief interview with the superintendent, he started with the foreman to see the finishing mill. The foreman seemed to understand perfectly what information was needed, and, while he told all that was necessary, it is not recalled that he used a superfluous word. He went about the mill rapidly from one place to another, and, after covering the ground, stated that the rails in question would begin to come out at a certain time. There were fourteen straightening presses and seven mill inspectors. The rails were to be loaded as rapidly as finished, and one can imagine how many of them a single foreign inspector could see. But when the rails began to come out, instead of trying to see all of them, he watched the mill inspectors. At times he walked and lined a bed of rails and tried the gauge. He observed the loading and looked over the loaded cars. Occasionally, he found a defective rail, and, by keeping busy, he was able to observe the attitude and efficiency of the mill inspectors. The result was most gratifying. There was evidence everywhere of the most perfect system. Every man was on his mettle. The superintendent had control of the mill. His wish was law, and every man, apparently, did his best according to his own skill and opportunity. Some did better work than others.

None was perfect, but it was plain that each man was striving for good results. There was no discussion, no argument. Everything was done as nearly right as possible, and, although but a small percentage of the rails was examined by the writer, he was never more satisfied with any mill inspection he has had to do. The evidence of a skilful and determined effort to secure the best results was so strong that, in so far as their surface was concerned, he felt confident that no better rails could be secured anywhere.

Had the character of the inspection by the mill men been less satisfactory, no individual efforts could have avoided the loading of many bad rails, and, assuming the lot was not bad enough to reject as a whole, the dissatisfaction of a good customer would have been the manufacturer's only detriment.

Some men have discovered that uniform excellence of material and workmanship is the road to both cheapness of production and a generous demand for their products. They do their own inspecting very thoroughly, and are well pleased with the results. It is a pity that there are not more men of the same mind.

There is this to be said, however, in favor of the contractor doing general work. He meets many engineers with many different ideas of construction, and if he is himself an engineer by training, he finds at times that the things he would do as an engineer do not please the engineer in charge of his work, and the things which the engineer does require are things which he as an engineer would condemn. After a few experiences of that sort, it is not to be expected that a contractor will do more than enough to secure his compensation.

When the contractor exhibits an unwillingness to perform good work, or to replace defective material, the task of an inspector becomes very difficult and at times unpleasant. To prescribe a set of rules for bringing a contractor to terms would be as absurd as it is difficult. When the conditions of strife degenerate to a state of war, only a soldier can understand the multitude and variety of tricks and devices which are used to defeat the aims of proper inspection; and, to meet these attacks, to checkmate the moves and emerge from the contest with a clean record and a good reputation, requires the resourcefulness of a modern diplomat. In all such cases, the inspector is not only obliged to keep his temper, but he

must be just. Recrimination is fatal. It places him on a level with his opponent, and subjects him to the same examination and suspicion. He must maintain a judicial attitude, and permit no irritation or fear to influence in any way the discharge of his duty.

When some engineers find that a contractor is disposed to slight his work, they permit him to operate for a time unwatched, and then, appearing suddenly on the scene, catch him red-handed and require him to tear down and rebuild the unsatisfactory work. It is natural to feel resentful and to wish to retaliate for a breach of good faith and confidence, but the law does not uphold such methods. It is the duty of an inspector to be on hand while the subject of his inspection is under performance, and, if he absents himself, he is at least morally guilty of contributory negligence, and should be estopped from exercising the same rigidity of inspection that he might use were he himself not at fault. In other words, being partly responsible for the poor work, he is not in a position to pass judgment and inflict punishment upon the contractor.

The time to protest or condemn is always the time of the act in question, and, in law, a man who remains silent when he should have spoken is regarded as a partner in the guilty act.

A man who protests habitually against everything that is done becomes known as a "kicker," but, however disagreeable that appellation may be, it is better to call attention to any defect at once, speaking to the contractor first, and if the matter is ignored, it should be referred to higher authority or taken under consideration by one who has power to act.

But when a protest concerning work or material is ignored, the inspector should not sit idly by and see all evidence of the defects obliterated, so that when the time of settlement arrives there is nothing to support his statement. Should the law be invoked to settle the matter, his word would not go far without confirmation, and he should preserve for future use such notes, references, or photographs as would enable him to prove, absolutely, the correctness of his assertion. The contractor, of course, would seek to rid himself of such an "officious inspector," but these things verge upon war, and a man must be the judge of his own acts. If he is a man of good morals and the right preliminary training, his wits will help him through many a scrape, and it is one of the pleasures of

an inspector's life to receive expressions of gratitude for kindnesses done to a contractor who has all but accused him of crime.

One of the most useful things which can be done by an inspector of field construction is to keep a daily journal of the progress of the work, carefully noting every occurrence which may have a bearing on the settlement of the contract. Such a journal, when supplemented by photographs, is not easily controverted, and will often clear away disputes which might otherwise be taken into court.

It is generally understood in the business world that expected profits should increase with the risks involved, and general contracting is considered a somewhat hazardous business. The risks involved are due to the elements, physical difficulties encountered, fluctuations in the price of materials, labor difficulties, and many other things, among which the character of inspection is not the least. To many people it will seem impossible that, with honest work, the character of the inspection may vary enough to make a material difference in the cost of work. It is true, however, and inspection is a factor that often enters into estimates for proposals. In some classes of work, the range of inspection, from the poorest to the best, is so great as to exceed the contractor's profits. This happens where the work is done on a very small margin and the contract is vague and indefinite.

To reduce this risk in engineering work should be the aim of every inspector. Perhaps the financial saving due to a careful, uniform and systematic inspection would be of less value than the elevation of morals surrounding the work. Selfish and dishonest motives are so common that the press and the public need only to discover a place where an illegal revenue is possible, in order to raise the cry of graft. In a practical way, confinement on suspicion is fully as inconvenient as confinement on conviction, and it is both practical and spiritual wisdom to avoid the appearance of evil as well as the real thing.

This enters the realm of professional ethics, which is ever open for discussion. Differing views are held about the propriety and also the morality of advertising oneself as agent for a particular brand of cement while holding a high public office involving control of large masonry constructions. It is conceivable that such a condition might exist with no moral injury to the principal, but the de-

moralization among his subordinates and the suspicions of the public would do great harm, and that harm would be centered on the engineering profession. In such a case, the wisdom of avoiding the appearance of evil is obvious, and it should be equally plain in matters of lesser importance.

On the other hand, a man may be actually responsible for much serious mismanagement without being amenable to the law. If there be no evidence of moral turpitude, conviction is impossible; but, to a professional man, there is punishment other than that of the law. The loss of prestige and reputation, gained by many years of earnest labor, may not be less disastrous in its consequences than a term in prison.

There is much said about the relations existing between the parties to a contract and the engineer, it being generally held in the profession that his attitude should be strictly impartial and that he should be no less alert to guard the interests of the contractor than those of his employer. Such a condition is a pleasing fiction, quite flattering to the engineer and agreeable to the contractor. It is conducive to harmony and good feeling, and is morally elevating; but, from the contractor's standpoint, or from a legal point of view, in what way can the agent of the party of the first part conserve the interests of the party of the second part when his sole compensation is received from the first party, and without which compensation he might be in danger of hunger? It may be true that some good and noble men have at times defended the rights of the contractor, and, by so doing, have placed their own incomes in jeopardy, but such cases do not make the rule. Engineers are not saints, by any means, and a sensible contractor will look sharply to his own interests where the elements of uncertainty involve large sums.

The real point in question is the clause in the contract which imposes certain duties upon the engineer. In so far as these duties are definite, the engineer should be absolutely impartial, and, to his glory be it said, he generally is. In fact, so far as the measurement of quantities is concerned, the whole system of contracting is based upon the honor and integrity of the engineer. Many thousands of dollars are paid each year on estimates of the engineer which are taken on faith, seldom being questioned unless the con-

tractor is losing money. Engineers are generally honest, but it would not be well for contractors to depend very largely on their assistance. Their duty is to conserve the interests of their employers, and they have the credit of doing so very successfully—in fact, so successfully that their employers, the world over, are the multi-millionaires, while the engineer—has any one heard of a multi-millionaire engineer?

There have been times when a contractor, engaged upon work of unusual difficulty and of great importance, has found himself in financial trouble and has been aided in some way to finish his work. He may have been given more work at a better rate, or he may have been given extra compensation. Such things are done, not in the nature of generosity, but because, in an emergency, it has seemed the best way to keep his plant and force at work and complete the undertaking. One should not get the idea that such things are gratuities.

This is a good place to point out an essential difference between public and private work. Contracts with municipalities or other forms of government are entered into in pursuance of law. All conditions surrounding the receipt of proposals and the letting and performance of contracts are provided for and governed by law. In such cases the duties of the engineer are very explicit, and neither he nor any other individual has authority to vary the terms of a contract in the least. If changes become necessary, the legal provision therefor must be strictly followed. It may happen, and sometimes does, that the engineer in charge of public work assumes authority to make changes and does make them without any serious consequences, but that fact does not alter the case. He makes the changes on his own responsibility and at his own risk. If he has erred in judging the temper and forbearance of the people, his reputation may be ruined, and he has no defense. The law is against him, and only his friends will believe him innocent.

This has been a stumbling block for the engineer in leaving private work to enter the service of the Government. The liberties which he took with contracts, when only his principal and the contractor were interested, cannot be repeated with impunity in the Government service. It is necessary that all changes shall be performed in a stipulated and formal manner. This is a safeguard against

corruption, and to ignore it is to raise a presumption of fraud. "Helping out" a contractor on Government work has brought more than one engineer to grief, and this difference should be remembered.

No inspection can be of much value unless supported by power to secure what the inspector demands. As we have seen, this power should be provided in the contract, and is vested primarily in the party of the first part. The power of the engineer lies first in his knowledge of the subject and his ability to demonstrate the injury that may result from a failure to carry out his plans.

A secondary power lies in the custom of making payments subject to his approval. He can, by withholding payments, be sure that his protests will be heard; but it must be assumed by the inspector that power exists to sustain his action, and to the full support of that power he is surely entitled. It is a waste of money to keep an inspector on a job if his reports are not to be heeded, but it is also far worse than that. If the contractor finds the inspector has no support, his presence will be ignored, and the work will suffer. Were there no pretense at inspection, the case would not be so bad, but, in placing an inspector on a piece of work, the engineer says in effect to his client, "Your work is under close supervision and will be properly done." Then, if he ignores his inspector, he deceives his client.

Owing to the imperfections of human life, it will always be true that, for one reason or another, some inspectors are incompetent, but their cases should be exceptional. It is fair to presume that the inspector knows his business, and, in employing him, the engineer becomes his sponsor. That inspectors are discharged and transferred is not always their own fault. Lack of moral courage, ignorance of the necessities of the work, and inability to direct the work of others are some of the faults which engineers display in handling their inspectors. Lack of appreciation is another. Few things will do so much to aid and encourage an inspector as the knowledge that his superior is familiar with and pleased by his work. The inspector's failure to please is often due to the fact that his chief has failed to keep in touch with him and let his wants be known. Perfect loyalty is expected from an inspector, but such loyalty requires a confidence which is the result of close personal relations. In Government work, especially, where it is often im-

possible for a man in a subordinate position to tell who has the real control of his department, an inspector stands in especial need of a strong and fearless superior, who is not afraid to do what is right and knows how to have his orders obeyed.

The inspector is frequently thrown in closer contact with the contractor than with his own employer, and finds his friends in the contractor's camp. Under these circumstances it requires a high order of character to be at all times faithful to his trust. The difficulties and temptations surrounding his life are great, and he needs his employer's full moral support.

An engineer who will seek to learn from the contractor whether the inspector is a good one, or who will reprove or over-rule his inspector on complaint of the contractor, has little knowledge of the duties of his office. There should be no occasion to over-rule an inspector. It is the business of the engineer to foresee and prevent it. Until he is sure of his inspector, he should keep in such close touch with the work as to prevent a conflict.

It is a curious fact that much of the friction which arises because of inspection is due to little things about which the contractor cannot afford to fight, and the question arises, why does he do it? The answer is believed to be that it is a question of personality rather than a question of workmanship. If men cannot agree they will fight, no matter how small the importance of the issue; from which we immediately conclude that the all-essential requirement of inspection is tact. Tact is the "Open sesame" to the success of the constructing engineer and all his assistants. A great engineer without tact is in many ways a failure. With tact, he could conquer the world.

In looking back over the subject, an estimate may now be formed of the preparation or training desired in a prospective inspector. As the term has been used somewhat loosely, to apply to engineers in charge of construction as well as to men whose duties are confined to a specific subject, it is necessary to discriminate. The engineer, of course, must have a technical training, but his education should be broader, embracing something of law and of business methods. These things he may acquire by experience. He should be brave morally, never hesitating about a question of honor, and he should be just and honorable in all his business relations. To

these qualifications he must add a liberal supply of tact. Without it the struggle is too hard. He might better expend his energies in some other direction.

An inspector may or may not have had a technical education. It is always an advantage, but seldom essential. He needs principally a few years of experience in the affairs of the world, a fair degree of intelligence, unquestioned honesty, and a generous amount of tact. An effort is sometimes made to get a mason to inspect masonry, a woodsman to inspect ties, and so on. Such a policy is wise at times. Very often it is not. The plans for engineering work so often vary widely from the local practice in various callings that preconceived notions are a detriment, and detract from one's value as an inspector. An inspector's mind should always be open to receive new instructions. He should not answer "Yes, yes," to questions about his understanding of a given matter, and then govern his actions entirely by his own experience. He can much better dispense with the previous experience.

Inspectors trained directly in the work they are to do are most desirable, and the best way to get them is to train them. For this purpose, the young technical graduate is without a peer. His education enables him to grasp readily the fundamentals of the subject; his mental attitude is that of one ready to learn, and the ambition and energy which have enabled him to complete successfully a difficult course of study will enable him to overcome most of the difficulties of inspection. Such men also outrank the average man in integrity and honor, and have a breadth of view that enables them to comprehend the whole situation. Their greatest lack is tact, for where the mind is steadily concentrated on study for too long a time it does not seem to grasp the common affairs of everyday life. This deficiency is sometimes made up very quickly and sometimes not at all. Without tact, a man should not be an inspector. With it, he need have no doubt of his success.

DISCUSSION.

JAMES SMITH HARING, M. Am. Soc. C. E. (by letter).—The author seems to lay particular stress upon the "tact" which an inspector on construction should possess. The writer agrees with him, but would emphasize equally the fact that an inspector must be, also, thoroughly honest and thoroughly loyal to his chief or his principal. The writer has been a sufferer in cases where an exceptional amount of "tact" in his inspectors and an absence of honesty and loyalty have not only suffered the work to be compromised, but his own position to be jeopardized—the "tact" working for the interest of the contractors. Mr. Haring.

The personal equation of contractors is as material to courteous relations on work as the personality of the inspector. Some contractors are honest in their intentions and faithful in the performance of their obligations, no dereliction of duty or performance is a part of their creed, and yet they may have in their employ men actually performing the work who think they can perform or omit the performance of certain obligations in a manner that will save money for their employer, thereby advancing their own prospects. Necessarily, a careful inspector objects to such actions, and, unless the contractor is himself honest enough to see the justness of criticism, friction must arise, and the result soon determines the relations between the engineer and the contractor.

Chief engineers, themselves, are not always reasonable in their relations to either their inspectors or the contractor. The writer had occasion to observe this not long ago. Through a series of misfortunes, a contractor was doing a piece of work at a considerable loss, financially, but, notwithstanding this fact, when any reasonable request was made by the inspector for any particular item to be executed, it was done with alacrity and cheerfulness, the whole aim of the contractor and all his men being apparent in an attempt on their part to do the best work possible regardless of the cost. With all this evidence of good intention before him, the chief engineer, in several instances, cast suspicion on the work, both of the contractor and the inspector, in a manner so aggravating (and affecting matters so trifling) as to arouse the antagonism of the inspector, who was both honest and loyal, and the contractor, simply because it was unjust; yet the work, when completed, received the most positive commendation as being first-class in every particular, and it certainly deserved such praise.

On the other hand, the writer has had certain adverse experiences with contractors. In one particular instance the contractor started out with the avowed intention of laying all manner of pitfalls for the engineer and his inspectors, intending to catch

Mr. Haring. them sufficiently at fault to find cause for making the contract void and laying the blame on the principals, behind the engineer, who acted upon his advice. In two other cases the incompetency (inexperience or financial inability) of the contractors was shown at the very beginning of the work, and it required not only "tact," firmness, and a knowledge of contract law, but a fight against political influence in the camp of his principals to enable the engineer to guide, not only the details of the work, but the persons who were paying for the work, out of grievous and sore tribulations, and to save his own reputation.

Too much cannot be said regarding the knowledge of the law of contracts to be possessed by the engineer. He must be able to protect his own rights, and unless acting for some corporation which maintains a legal department or retains regular counsel, he should be able to advise and direct those for whom he acts. In any condition, the engineer is charged with large responsibility; he is the active agency to take the responsibility for the work of his inspectors and assume it as his own; he is the target of the contractor and all his employees, and must stand their criticism and too often their abuse; he must frequently meet a combination of the contractor and his own employer or principal trying to place him in the wrong; so that his justification and defense must stand on indisputable grounds of right, or he is likely to fail utterly.

The condition which makes the engineer sick at heart, however, is to find honest effort on his part met by censure and condemnation in the light of the most positive evidence of the fact that his course is correct. The writer recalls a condition where he, as engineer, called to his aid inspectors whose capabilities, honesty and loyalty had been proven on other work. These inspectors did their duty against adverse circumstances. The work was for a municipality, and he regrets to say that he has spent whole sessions with the governing officials, who took up the time, badly needed for important business, in listening to the complaints of inexperienced contractors against these inspectors, simply because they had performed their duty conscientiously. The writer upheld them, and refused to countenance censure or removal. Finally, he himself was replaced by a more pliable man, who permitted the contractors to obtain payments for extra work to which they were not entitled, and who permitted the use of materials the writer had rejected.

In another instance the writer was engineer on a contract abandoned by the contractor because the engineer ordered an additional quantity of work done at a specific price named in the contract. The work was finished by the municipality under the terms of the contract, the writer acting as engineer, and a large deficiency for cost of construction has just been obtained by a jury trial in a suit to recover from the contractor.

Under any conditions, the engineer and his inspectors occupy responsible positions. If due consideration is not given in the selection of the inspectors, and there is not sufficient latitude, or absolute control of appointment, it too frequently occurs that competency is not the determining factor in the choice of men. A contractor is very soon able to "size up" the capabilities of the man who is to watch his work, and he acts accordingly. Mr. Haring.

As this is the age wherein women seem to be displacing men, the writer has often wondered whether the time will come when, to some degree, women will displace men in the engineering profession. In certain cases mentioned herein the proverbial "tact" of women might be found to be advantageous.

W. D. Lovell, M. Am. Soc. C. E. (by letter).—This topic is one of general interest to both engineers and contractors. Undoubtedly, "there is too much laxity in the preparation of specifications." It has become the custom with many engineers to write their specifications hurriedly, or to have them prepared by an incompetent assistant, and then, to make sure that they have covered everything, to put in a general clause, as pointed out by the author. This blanket clause is an admission of the weakness of the specification, and its use should be discouraged. In making up his bid, the contractor depends upon the written specification to describe the work he is expected to perform and not upon the "general conditions" which may mean nothing, or a great deal, depending on the inspector. Mr. Lovell.

In one instance, coming to the writer's notice, the specification relating to concrete seemed to be voluminous. Omitting the specification on cement, one thousand words were used in describing concrete, yet, when construction began, the inspector found it necessary, in order to get such work as he thought desirable, to hide behind a general clause, applicable to all divisions of the work, which stated that "all work must be done in a workmanlike manner." Relying on this blanket clause forced the contractor to increase the actual cost of the work 50%, and no extras were allowed. It is injustice like this which causes unpleasant relations between inspector and contractor, and brings the engineering profession into disrepute.

It often happens, especially on Government work, that the man who writes the specification has no part in the supervision of the work. The opinion of what is a "workmanlike manner," on the part of the man who wrote the specification, may differ widely from the ideas of the constructing engineer. The man who wrote may have had in mind a good class of commercial work with nothing fancy or finished about it. The inspector may happen to have decided opinions regarding the finishing of work, and may attempt to compel the contractor to do the work according to his ideas. He cannot do so by the specification on that particular subject, therefore

Mr. Lovell. it is necessary for him to call up the "general conditions," usually written in specifications, that "work must be done in a workmanlike manner" or "to the satisfaction of the engineer." This is not a fair interpretation of the specification or the contract, and yet, as is suggested by the author, a contractor very often submits to just such injustice because he cannot afford the time or the money to take the matter into court. Each specification, covering any one part of the work, should be written so that it should not be necessary for the inspector to rely on general clauses to support his requirements in regard to the character of the work, and "blanket clauses," which may be used as a gross injustice to the contractor, should be omitted, except to cover unforeseen contingencies.

Mr. Himes has mentioned the necessity of the inspector knowing his business, and being familiar with the conditions surrounding the contract. The successful inspector must have a practical knowledge of first-class work. The contractor often repeats the old saying, "I have no trouble with the inspector who knows his business." It is the man who is afraid of his own position—who does not really know whether he is right or wrong—who, by his unreasonable requirements on some points and his laxity on others, shows his lack of knowledge, loses the respect of his chief and the contractor, and proves a failure as a constructing engineer.

The following is a case in point: The work consisted in laying cast-iron water pipe. The inspector was diligent in chipping and tapping the castings, although he had a certified test of the material used in the pipe and a sworn statement that each length had been tested under a pressure of 300 lb. He was careful to see that the letters, indicating the manufacturer and the year in which the pipe was cast, were easily legible, and that the dust was wiped out of the pipe before it was laid; yet, when it came to the really important work of seeing that the pipe had a good uniform bearing in the trench, that the jute was driven back, and the lead joint deep, properly run, and thoroughly caulked, he paid no attention whatever.

A part of the line was through a corn field, where any settlement of the back-filling would make no difference, as the field would be plowed over in the following spring. The back-filling was to be done with a scraper, the earth wet down with water furnished by the city, and pumped at considerable expense from a deep well. The inspection of this back-filling was the most severe that could be imagined. The additional cost to the municipality, on account of pumping water alone, was considerably more than the salary of a qualified inspector.

Mr. Thompson. BENJAMIN THOMPSON, M. AM. SOC. C. E. (by letter).—Mr. Himes' paper treats of a very important subject, and the writer, without desiring to be critical, wishes to state that it is not so much fluency

of diction that is needed in writing specifications as plain, definite, specific, concise, comprehensive statements. It sometimes happens that specifications are what might be called a literary effort, which is apt to confuse and bemuddle the contractor, who, more often than not, has had no more education than what is embraced in the "three R's." What he needs is plain writing, showing clearly and specifically what he is to do, when he is to do it, and what and how he is to be paid for it. If the engineer has not investigated the situation and the conditions surrounding the proposed work in detail, he cannot prepare fully what the contractor ought to have for his guidance. The more care in preliminaries the less difficulty with contractors, and the less embarrassment in explaining why the total cost exceeds the estimates, the latter being the standing indictment against civil engineers. Mr. Thompson.

The author's epigrammatic statement that "confinement on suspicion is fully as inconvenient as confinement on conviction," "should be indelibly engraved in the memory of every young engineer," as Mr. Wellington would have said. And the writer would like to add Whittier's "Ah, what a thin partition shuts out from the eyes of the curious world the knowledge of evil deeds done in darkness."

The author's question, "Has any one heard of a multi-millionaire engineer?" reminds the writer of what was said to him some time ago by a gentleman who had just paid $40 000 commission to a real estate agent for the sale of some property. "That man worked two or three months, and an engineer might work as many years or more to get the same amount."

The writer asked, "Why should the real estate agent receive so much higher pay for his services than the engineer for an equal or much larger amount of work to make the sale possible?" His reply was, "I don't know, unless it was because he got the money."

Inspectors or resident engineers on extensive work usually come from different sections of the country, and have different ideas as to good construction, and how to proceed in special difficulties. It seems to the writer that it would be a good plan for the chief engineer to have the engineers and inspectors come together at the beginning of the work and at various times during its progress for the examination and discussion of the specifications and their application to the special difficulties or problems which may be met. If this were done, the character of the work under their charge would be more uniformly good, contractors would feel that they were all treated about alike, the individual inspector or resident engineer would feel that he could take a position in which he would be sustained by the judgment of his comrades and his chief, with the least annoyance and trouble to the latter, and the chief engineer would

Mr. Thompson. be doing most valuable service, not only to his employers, but to those under him.

Mr. Russell. S. Bent Russell, M. Am. Soc. C. E. (by letter).—This subject covers much of the whole field of engineering, and so much may be said upon it that it is not easy to treat it well within the limits of a brief essay.

The broad question of the proper interpretation of engineering specifications is of such importance to engineers that it might well be classed with those subjects that should be brought up annually for discussion.

Consistency is of great importance, both in writing the specifications and in executing them. Not only should an engineer be consistent in his own practice, but neighboring engineers should advise with each other, and try to make their practice consistent.

Going a step further, there should be a certain amount of consistency aimed at among the members of the whole profession.

The dictum of the engineer should gain strength, and, indeed, often does gain strength, by the fact that it not only represents the judgment of a man especially trained for the work, but also in a measure represents the consensus of opinion of the whole engineering profession.

The standing of the profession will certainly be elevated by greater unity among engineers in the matter of engineering contracts. A great deal of missionary work is needed among engineers. They should be better informed as to what is the best practice, both in writing and in interpreting specifications.

The author is to be congratulated upon his successful opening of so important a discussion, and upon the many good points that he has brought up in the paper.

As to preparing specifications, engineering has now become such an advanced science, and the volume of engineering knowledge has become so vast, compared with the capacity of any individual, that, with little violence, it might be said that when an engineer starts to write a specification, he is working on a subject of which he has little or no personal knowledge. For example, he may have had much experience in building structures of steel and of masonry, and now be called upon in the line of the structure of timber. He is dependent upon the work of other engineers. His information must be gathered from books, periodicals, etc.

Perhaps the first question for the engineer to decide is how much time can be given to research work. It depends, of course, upon the importance of the case. Most errors in engineering specifications come from lack of time and information in preparing them.

In the writer's judgment, engineers more often give too little time than too much to specification writing. Many a lawsuit could

have been avoided by a few minutes more time in the beginning. Mr. Russell.
This is a point that could be thoroughly discussed by engineers with great advantage, and the position of the author is well taken as to better specifications being needed.

Perhaps the next question, with the engineer who starts to draw up specifications, is the state of the market. If contractors are known to be hungry for work, specifications, of course, may be severe. They may be written so that the contractor will be lucky if he gets his capital back.

On the other hand, if contractors are well supplied with work, they are usually very independent. The engineer must see that the specifications are more than fair to the contractor. Otherwise, he will get no bids, or excessive prices.

Coming now to the inspection of engineering work, as in the drafting of specifications, it is found that the greatest difficulties come from ignorance. Young inspectors, of course, do not know what is customary practice in the line of work which they are called upon to judge. In their inspection, they are often at a great disadvantage, in that they know less about what is right and customary than the contractor himself, and yet they must expound the law and control his actions.

On the other hand, inspectors of more experience are often worse, on the whole, because they have been badly trained. An inspector who has served under a careless or incompetent engineer may be very unsatisfactory, owing to faults in his training. An old bricklayer does not always make the best inspector of brickwork. He is apt to be prejudiced by the kind of work he has been doing in his trade.

The number of inspectors or the quantity of work to be covered by one inspector is a matter of great import. One does not like to have the cost of inspection too high in proportion.

Engineers, as a class, should insist that the success of the work should never be imperiled by fear of criticism as to the cost of inspection.

At the outset the inspecting engineer may well bear in mind the state of the market when the contract was let, as mentioned previously, and thus infer the probable width of margin on which the contractor has to work. The keener the competition the sharper should be the inspection, as a rule. More often than not, the amount expended on inspection is below the point of greatest economic results.

Specifications which give arbitrary power to the engineer are too apt to put him in an improper state of mind. With the best intentions in the world, an engineer unconsciously becomes careless of the rights of the contractor and sub-contractors until he is con-

Mr. Russell. fronted with a lawsuit. The engineer and inspector should guard carefully against this failing.

Perhaps the most important point mentioned by the author is "the practice of helping out the contractor." In the writer's judgment, the views of engineers on this point should be fully brought out, and a great effort be made to get the consensus of opinion of experienced engineers on the question. There is too much variation in the practice of engineers in this direction. Personally, the writer thinks that, in any work of a public nature, the engineer should never permit "helping out" the contractor. Obviously, it does no good to contractors as a class, and better results will be obtained with the practice ruled out altogether.

In private contracts, this practice should never be allowed, without full knowledge of all the parties interested.

It should be noted here that there is often a third party, not named in the contract, but interested in the result, financially or otherwise, and this party is depending upon the fairness and integrity of the engineer to protect his rights and see that the work is executed as planned and brought to a successful conclusion.

The engineer should keep the interests of the third party in mind, and guard them where it is proper he should, or advise such party of any proposed changes in the work.

A matter of similar bearing is the proper treatment of subcontractors. The engineer should be fully informed as to the status of such parties, and should know whether they are being fairly treated by the general contractor; and, where their interests are at stake, should keep them in mind, carefully avoiding just cause for complaint from them.

An error that the engineer should guard against especially is that of allowing the contractor to deceive himself by practicing a sort of confidence game upon him. The contractor is allowed to believe that he is to be "helped out" by the engineer at some later time, and thus he is kept in good humor and tractable, without the engineer really committing himself. This is sometimes the engineer's method of managing the contractor, but it is decidedly dangerous, and should not be followed.

Engineers should settle disputes as soon as possible, and should not defer them. There is room for a great deal of diplomacy in the management of contractors who are engaged upon different parts of the same work, so that each shall be made to think that he is being treated at least as well as any other contractor. To do this, under all the complications that come up in engineering work, is sometimes most difficult.

At times, where a contractor shows a disposition to make trouble, the engineer will make concessions to him that are of doubtful pro-

priety, in order that, should a lawsuit come up later, he may show that the contractor has been treated liberally. This, of course, is treading on dangerous ground, and the engineer should be very careful not to go too far in this direction, as his motives may easily be misconstrued. Mr. Russell.

The question of covering up errors made by the engineer in writing specifications, or in the plans or instructions, is one that could be discussed with interest. How far may the engineer go with propriety in this direction? This question often arises in actual work, and is frequently of great import. It is doubtful, however, if any generalizations can be made for such cases that would not apply to the conduct of human life.

The writer hopes that the points thus briefly called to mind will add something to the interest of the discussion.

Willard Beahan, M. Am. Soc. C. E. (by letter).—This paper is an unusual one in both its field and scope, and will be useful to the profession. Engineering has been broadening much of late, and annexing new territory to the old province of mathematics and science. This paper is of the new learning upon which engineers are entering. It trenches much on law, and it has to do with human nature. In each direction an engineer always needs instruction, and to-day more than ever. Mr. Beahan.

The author states that "A contract is a meeting of minds," and that there must be a "clear and definite understanding of what is to be done." Now, this understanding of the two minds is the contract. The written instrument is but a reflection of it, and is sometimes clouded by legal phrase or ignorance of the niceties of technical terms. That printed blank which we fill out is not the contract, in the eyes of the law. This fact makes blanket clauses a confession that the minds have not clearly met, and hence the contract is faulty, and the door of misunderstanding and of litigation is confusedly thrown open.

Engineers differ as to whether a detailed contract and specifications which endeavors to cover all points fully and minutely is best; or whether a brief and general specification is best. The writer supposes that each engineer will ever favor that one of these two which is better in accord with his own temperament. Governments and municipalities favor the first kind, while corporations and firms very often favor the latter. Most engineers will agree that, in general, during the execution of work under contract, questions will arise which are at least on the border line of the terms of the contract, and will raise questions for adjustment. The engineer in charge is the one to whom these questions present themselves, and they are a crucial trial to him. With reference to these recurring questions the writer's instructions at the outset of his career, as

Mr. Beahan. given to him by the late D. W. Washburn, Chief Engineer of Construction on Mr. Gould's South-West System, were as follows: "You are, of course, employed and paid by the Company, but it is your duty to stand between the Company and the contractor, and say what is right in equity." Thirty years have almost passed, but to the writer the instructions still seem to be right. Common sense is good railroading, and honesty is the best policy, as well as the best politics. No contract can be drawn that makes the exercise of judgment unnecessary.

As engineers, we cannot compel a contractor to do work not really shown on the plans or in the specifications or in the contract. Some contractors have done harm to engineers by willingly doing such work just to cover some sins of omission. The whole is vicious. If a mistake is made and some work is left out, say so; it is better for all, and cheaper for the company, to handle this as extra work outside the contract. The engineer or inspector who accepts favors from the contractor is lost. Blanket clauses are put in contracts to cover just such careless work, but anyone at all familiar with the law of contracts knows very well that such attempts are futile, and this fact is clearly brought out in the paper. It is to be hoped that the quoted clause in the elevator and power-plant contract is the very limit of unwisdom in this direction.

"Absolutism has no place in business." One would think this were not true if he read some contracts, where it would seem that the engineer can do anything he pleases and pass any judgment he sees fit. But the courts sometimes make short shrift of such contracts, and it is readily seen that a "meeting of minds" has not yet convened. In fact, a "printed form," when used in a contract, biases a jury and brands that contract as arbitrary. As a profession, we need to know how courts and juries view us and our acts. We need to get out of our offices and drafting rooms and just "come down to earth" and mix with the multitude. By doing so we can better earn our salaries in our mature years.

It is indeed hard to say what is 89% and what is 91% when 90% is the mark at which inspections are passed. As inspectors, in our younger years will we not mark it 89% and in our later life mark it 91 per cent.? Can we ever detect that 2% of excellence? Clearly, we must use our honest experience and judgment. But to be called back and shown that some of our 89% product is as good as some of our 91% product is the act of a young or ill-tempered contractor. An old Irishman of some experience and sound heart was once inspecting cross-ties and was shown some rejected ties on one side of the pile that were a little better than some accepted ties on the other side of that pile. After a moment's study he went back and rejected them all. Did he do right?

Mr. Beahan.

The absent inspection of masonry, like the absent treatment of disease, is believed in by some, but they are in the minority. There is too much inspection which does not inspect, and it brings inspection into disrepute. To have one inspector over two gangs of masons five miles apart is as foolish as to have one foreman over two gangs. This evil arises from letting too many small contracts when the cost of inspection thereupon adds much to the unit cost. A better plan is to do these small jobs by a company force.

The case of the mill inspection of rails cited in the paper raises this question, naturally: May it not cost less to try to make first-rate rails than it will cost to make poorer ones and take the chances of their rejection? Some may have seen grading contractors haul logs and brush into an embankment when it would have cost less to burn the logs and brush and haul in earth instead. Cheating comes to be a chronic disease at the last. The writer once knew a contractor to tell a falsehood wilfully when it was against his interests. His brother, who was the other member of the firm, said, "W. is a fool! W. will lie at sixty cents on the dollar when the truth would be worth par." We can form the habit of thinking that the truth is ever against us.

An inspector, not sustained or "backed up" by his superior, is a man whose salary is money wasted through no fault of his. Honesty, capacity, and courage are essentials in inspectors, or in engineers in immediate charge of the work. Said a prominent chief engineer of a granger road to the writer a few years ago, "I have had two hundred and forty civil engineers of various grades on our work this summer and in not a single case has there been the least suspicion of dishonesty." In a quarter of a century of railroad work over much of the United States the writer has never seen a dishonest act by an engineer. He has had only one dishonest engineering employee—a paymaster, and under mitigating circumstances in a foreign country. Will our friends, the contractors, who are given to careless expression, please commit to memory these statements of the writer on this point? Capacity, however, is a rarer quality. The writer thinks that too much is expected of young engineers as inspectors. It is not fair to them. The writer fully agrees with the paper, but must say that, for an inspector of execution, rather than of manufactures, he prefers an experienced craftsman. For example, his best results in pile-driving have been in using an experienced pile-driver man as an inspector, rather than a young engineer. So, too, for a modern building, he prefers a first-class carpenter who can read a plan, rather than a young architect or engineer. For such work he prefers an old foreman, of unusual intelligence, who, perhaps, may be in poor health or crippled, but who possesses all his faculties. If he has been for a long time with the company, so much the better.

Mr. Beahan. Courage is a quality of character, rather than of education or experience. Years ago a collegian was not thought to be courageous, but college athletics have changed all that. Moral rather than physical courage is the kind most required, but they go best together in this case. Bluster is the outward expression of pure cowardice. The brave are quiet and use few words. Of these three great qualities it may be said that, as a rule, honesty is a matter of course with a technical graduate; capacity can be trained into him, while courage he must have inherited, in the main.

The author has properly pointed out the fact that tact is an essential to greatness in the engineering profession. Tact is not taught in technical schools nor could it be. But a little tact might have been talked to us now and then to good advantage. We were taught intolerance, and were sometimes taught self-sufficiency. This is not seen in the later generations of students, however. In later years engineers must learn that tact which they ignored at first. Tact is ready money where talent is capital. As a profession we are at fault in not cultivating tactfulness. It is the new learning of our calling and the younger engineers should take up the study of it right away. Knowing a thing is not enough. One must also be able to make others and capital know it, too, and through one's own self as the instrumentality. Learn a thing, learn to tell it or write it, learn to do this convincingly, and, finally, learn to do so in a pleasing way.

Mr. Aiken. W. A. Aiken, M. Am. Soc. C. E.—It is very refreshing to find in this paper a clear note of appreciation of the real value of inspection; not only of the actual features of any piece of engineering construction, but particularly of the materials entering therein. It is unfortunately the fact that many engineers do not fully realize the true value of this latter and certainly equally important inspection, or, if realizing it in a general way, have neither the time nor the opportunity to acquaint themselves sufficiently with its infinite detail, and so unconsciously confound the mere matter of testing with real inspection, of which testing is only a very small part.

In the speaker's opinion, there is nothing more certain, when once thoroughly grasped by contact with a properly organized system of inspection, than that no engineer would ever think of using materials in construction unless previously they had been thoroughly inspected; for, of all the specialties of engineering technical work, no other has been so generally brought into disrepute, with those "who understand," by this age's spirit of commercialism, the dominance of which has set up everywhere the false standard of dollars and cents. Thus the cost of so-called inspection, rather than the quality of service, is unfortunately very often the conclusive argu-

ment in deciding a matter which in no way should be thus in- Mr. Aiken.
fluenced. Inspection that does not inspect is absolutely worthless in itself, and is a complete waste of money. The mere matter of testing (relieving the manufacturer of responsibility, as it does in a great measure) is in many cases an absolute farce, in so far as determining the true worth of the material furnished, particularly because, as Mr. Himes states, it has unfortunately grown into a generally recognized practice that the tests submitted are to be of the manufacturer's selection.

The tonnage—in these days often involving a poorer quality of manufactured material—is another feature influencing largely the quality of inspection, unless this is carried out purely upon the basis of quality of service, instead of as a commercial enterprise. When the manufacturing plants are crowded with orders, each being pressed for prompt delivery, and, to meet these demands, the manufacturers are putting all their efforts toward increasing their output, with the temptation to disregard its quality, an entirely different condition confronts the inspecting engineer than when orders are few and competition keen. The purchaser demanding shipment, the manufacturer, knowing that among his customers there are some whose own or commercially employed inspection is largely perfunctory, though these very customers (due to their ignorance of what inspection should consist) may not realize this, and knowing also that others are perfectly willing to accept the manufacturer's guaranty, are features of the conditions which are difficult to meet except by the strictest insistence on the specifications in all essential matters, and the exercise of clear judgment in the matter of concessions, if it is desired to obtain material complying even with admittedly fair requirements. It is under such conditions that what Mr. Himes designates as "the kicker" develops, but the speaker is satisfied that this is desirable, although, in his opinion, it is not generally necessary, and never in the case of a competent inspector, except in plants in which the methods are not "straight." Certainly, by ignoring conditions at the start, involving, as this must, the quality of inspection guaranteed (and surely, thereby, the quality of material contracted for), the inspector becomes *particeps criminis,* making it more difficult afterward to protest effectually against aggravated and intolerable conditions, and making him largely responsible for the attitude of many manufacturers toward proper supervision by the inspector.

Competent inspectors, even when supposedly hyper-critical, are generally so only from the manufacturer's erroneous standpoint. The manufacturers may maintain that they are entitled to only such information and facilities as are supposedly customary and called for by the average inspector. This is not tenable. On the

Mr. Aiken. contrary, any information in the manufacturer's possession relative to the material to be inspected belongs by right to the purchaser's representative, and may be properly called for without his deserving the name of "kicker," even though his requests may be beyond those made by the average inspector. There is no conceivable reason for a manufacturer to refuse such information and facilities, as at times is done, except the one plea that it is not customary—and this is no reason at all—or the other reason that such information and facilities would enable the inspector to keep better run of the material, and this is worse than no reason at all. A manufacturer who has nothing to conceal never objects to furnishing any information relative to material under inspection. It is only in the case of questionable practices—and, of course, these are not general by any means—that the "kicker" is prominently developed, and for excellent reasons. Also, occasionally, where personal feelings enter into the business relations, the manufacturer deliberately and intentionally, as it were, develops the "kicker" by continually putting obstacles in the way of his properly performing his duty, with the object of bringing him into disrepute with his superior who cannot be personally acquainted with the details of every transaction. A conservative inspector can be sorely harassed by a petty-minded representative of a manufacturer, and the patience of a Job and the tact of a Talleyrand are then necessary to steer a course where duty is fully done and no opening is given for lawful objection to method.

The specious pleas of the manufacturer to influence an inspector even properly are many, and in a measure must be recognized from the former's standpoint as possibly allowable, but these must not influence. For instance, the manufacturer may request the acceptance of a small lot of rejected material on account of its size, which from the manufacturer's standpoint determines its importance; or he may request the acceptance of a similar larger lot of material because its rejection would entail a loss to him. He may also advance the additional pleas of busy times, the uncertainty when replacement can be made, and the always prominent plea that the construction will be delayed. Such arguments, in the case of structural steel, lose their weight in some mill practices; very bad ones they are, and in time they must be controlled, when it is understood that the test pieces furnished often do not represent any considerable quantity of material actually then rolled, though they are presumably the melts from which an order is to be rolled later, if the tests pass the specification requirements.

In the case of final rejection, the loss is really nominal at the worst, the material being applied to some other order where the specifications are less rigorous or the inspection less carefully made.

All of which emphasizes the fact that testing is not inspection, no matter how carefully and conscientiously the former may be done. Complete and thorough acquaintance with the process of manufacture in all stages, and the assurance that the material finally used is that tested, is the only criterion whereby to determine proper inspection, and to obtain this there is no information relative to the material that may not be properly asked for and required by the inspector. Mr. Aiken.

Under all the stresses to which he is subjected, nothing is more appreciated by him, nothing is more necessary to him, than the endorsement of his superior officer; and nothing, even from the most selfish standpoint, pays the engineer so well in securing the best service from competent men as the proper support of his inspectors.

While concessions may be properly made at times, in the judgment of a competent inspector or under general rulings of his chief, the idea that a reason must be found for the acceptance of material, whenever it fails to meet certain specifications, is too often the attitude of the manufacturer. The inspector in charge, knowing as he should, the use to which any material is to be put, must be the judge. Any other viewpoint is subversive and not to be tolerated.

It is to be hoped that the discussion of this most important class of work will cause engineers to recognize more clearly the value of proper inspection, which is undoubtedly of benefit to the manufacturer, who should have common cause with the inspector in producing first-class material.

Augustus Smith, M. Am. Soc. C. E.—This paper is considerably broader than its title would indicate. Besides the relation of the inspector or constructing engineer to his work, it opens up the whole question of that broad branch of engineering, so to speak, of how to get one's idea executed—how to get what you want done. The speaker has seen this problem from the viewpoint of the contractor—the virtuous contractor, he hopes it will be understood—and therefore will ignore the inspector altogether. Mr. Smith.

He will confine his few remarks to an idea that at first sight may appear to be irrelevant, but which has been suggested by two statements made in the paper. The first statement is on page 109, where the author says:

"The theory of the law is held by its devotees to be the discouragement of litigation, and, in this respect, because of the expense and the numerous difficulties and delays in getting a final decision, the legal profession has attained a degree of perfection which engineers may not hope to equal."

The law is so perfect in the direction pointed out by the author that, though every contract is based on ultimate recourse to the

Mr. Smith. courts, no one who has actually tried the process would think of trying again, even if he lived long enough. It is so perfect that those who practice it are making little or no effort to improve it. At the end of the last century, when, it will be remembered, a general *résumé* of progress in all lines and professions was rather a popular subject, the Law was the only profession that had no progress to report.

The second statement is found on page 118. It reads:

"There is much said about the relations existing between the parties to a contract and the engineer, it being generally held in the profession that his attitude should be strictly impartial and that he should be no less alert to guard the interests of the contractor than those of his employer. Such a condition is a pleasing fiction, quite flattering to the engineer and agreeable to the contractor."

This is indeed a fiction. Aside from the biasing influences pointed out by the author, disputes generally arise from poor specifications, and frequently from lack of knowledge by the man who prepared the specifications, who is then assumed to be impartial in interpreting them. What man, even among contractors, can be depended upon to be impartial under such circumstances?

In order to avoid the Scylla of the Court and the Charybdis of the Engineer, some contracts provide for arbitration in case of dispute. The speaker has had no personal experience with the working of the arbitration clause, but understands that in general it is unsatisfactory.

Now for a "remedy." If the American Society of Civil Engineers found it permissible and expedient to elect with the other officers a contract committee, having cognizance of such disputes as Mr. Himes refers to, much as the regatta committee of a yacht club settles all questions of fair sailing, a great advance in "arbitration" would be made.

The membership of the American Society of Civil Engineers includes many contractors and many engineers individually professional who are employed by contracting firms. A decision of the contract committee of the American Society of Civil Engineers would command more respect from this class of disputants than a decision on engineering subjects by the Court of Appeals.

Expert legal testimony might be necessary at times, but let it be the lawyer before the Bench of Engineers on purely engineering subjects, instead of the engineer before a Bench of Lawyers who are generally quite uninterested and frequently half asleep.

It would be necessary, of course, to provide a scale of fees for such a committee, properly payable by the contestants, and, if found desirable, these could be made high enough to "discourage" litigation.

G. S. Bixby, Esq.—Mr. Himes has taken up for discussion one of the most difficult and perplexing problems in the engineering profession. To outsiders the difficult features of the engineer's work seem to lie in logarithms and angles, in those terribly long lines of figures with signs of all sorts between them and over and under them, and in the mathematical features of the work generally; but, of course, those things are mere play to the engineer, and the speaker suspects that his real troubles begin when he is held responsible for money values, and when he is made a buffer between conflicting business interests. Mr. Bixby.

This paper is a very valuable contribution, from the standpoint of practical engineering. The speaker knows personally that Mr. Himes began turning these things over in his mind a good many years ago, for when the speaker first knew him he was an engineer on the Erie Canal, and even then had the idea that an engineer was supposed to work for the interests of his employer. Doubtless he had some troubles of his own growing out of the practical application of that idea. At any rate, some other people had troubles on his account. At that time he used to spend his spare time studying law, and he must have studied to good advantage, for his points of law seem to be generally well taken. One of the most valuable suggestions made in this paper is as to keeping a diary on inspection work. As a lawyer, the speaker will say that he has never seen a witness stumped on the stand when he had on hand for reference a record of events made in chronological order.

A prominent feature of the work of the constructing engineer is that there is a tendency in practice to exact from him what is well nigh an impossibility. Wherever engineering is made up largely of work which is soon concealed, or the evidences of which are soon destroyed, it would seem that neither the expense nor the time allowed for inspection is ordinarily enough to enable an engineer to give positively the certificates which are theoretically required from him. One cannot get away from the fact that the contractor's interest is not that of the owner; nor that the most honest contractor when pinched on his margins will exercise a tendency to pinch on his work. One must also remember that a contractor owes a duty to himself, sometimes to his creditors, and that sometimes he is a trustee.

Another point on public work, and often on private work, is that the certificating engineer has little to do with the choice of his inspectors. In a sense they are supposed to be his agents, but often they are independent employees, and it seems to be hard to exact from an engineer a certificate in the form which, as the speaker understands it, is ordinarily required that a thing is so and so as of his own knowledge, when he cannot see everything, and is dependent on others.

Mr. Bixby. As a matter of justice, an engineer responsible for results ought to have at least a partial voice in the choice of sub-engineers and inspectors.

This paper refers very intelligently to the subject of perfection in materials and workmanship. Of course, it is known that there is no such thing as perfection, and specifications ought to have incorporated in them the permitted variations more than they do. It would be simply following current practice, but, if that is so, why should it not be expressed? In public works the conditions are frequently quite inflexible. Sticks of timber and pieces of iron and steel must vary, and yet, under a standard fixed for Government work, every defective bolt may be a nail in your coffin if it comes to light.

For instance, where hundreds or thousands of units are combined in a structure it may be unjust to require every piece to be of a standard character, and yet on public works the contract, the specifications, and often a statute unite in fixing a standard, and when there are such plain, specific provisions it is hard to invoke the doctrine of reasonableness or current practice.

There is a general opinion that a professional engineer on contract work is a kind of judicial officer, that his decision is like that of a court, and that such decision must be made impartially between the parties. This is referred to in the paper on page 118. The speaker does not believe that this theory has any standing in law, but thinks it arises from the fact that, as a matter of law, when, under a contract, a question is submitted to an engineer or an architect for decision, his decision cannot be an arbitrary one, but must be reasonable. In other words, the contract is to do a certain thing in a workmanlike manner, and if it is so done the inspector is bound so to decide. So, although the speaker's experience has taught him that the engineer, in practice, is very apt to be treated as an umpire, in reality he is not an umpire.

Not only is he employed and paid by one side only, but in all ordinary cases he would be subject to discharge or transfer by one side and not by the other.

This opens a very broad field for thought. The engineering profession is increasing in importance daily. The great works are multiplying so fast that we lose track of them and a hundred million dollars is becoming an ordinary sum.

The public is becoming more and more inclined to undertake these vast enterprises, and, in doing so, it is more and more dependent on the engineer. The contractors' interests also require protection. On the face, the contractor would seem to be at a disadvantage, for, while the party of the first part can ordinarily call on the engineer for such protection as may be needed, the contractor,

if he disputes the reasonableness of the engineer's decision, must invoke the aid of a court. Mr. Bixby.

It is the greatest possible compliment to the engineering profession that so comparatively few lawsuits arise on important works. Of course, they arise often enough, and when they do they illustrate the difficult questions in hand.

The speaker has in mind one lawsuit now pending between a railroad and the contractor, which grew out of the holding up of final payments which would have been acceptable to the contractor at about $100 000. In the suit claim is made for $1 500 000 or more, and the contractor expects to make good his claim to many times the sum he would have accepted on the completion of the work.

Here is a question which often comes up: Suppose on important work a question arises which is obscure as to its solution, but nevertheless vital in the progress of the work. That is—the scope of the question is defined, it has to be decided, and engineering opinion relating thereto is divergent. Perhaps it has been in the courts and has been decided in different ways, or perhaps it has been left undecided. What is the engineer to do, and how is he to be protected? The parties can fight it out afterward, perhaps, but it places the engineer in a very difficult position.

Albert J. Himes, M. Am. Soc. C. E. (by letter).—It is a pleasure for the writer to express his appreciation of the kindly reception which has been accorded to a paper which, both in subject and in character, differs materially from those generally presented to the Society for discussion. He fully believes that the subjects discussed are of the most vital interest to all constructing engineers, and can be studied exhaustively with much profit. Mr. Himes.

Mr. Russell's suggestion that "the broad question of the proper interpretation of engineering specifications * * * should be brought up annually for discussion" is very pertinent, and is deserving of careful consideration.

The industrial development of the country has been so rapid that customs affording a sufficient basis for common law have hardly had time to mature; surely not time enough to meet the old English requirement that they shall have existed from "a time whereof the memory of man runneth not to the contrary." It should be the aim of the engineer, and especially of this Society, to work for the establishment of such customs, and thereby purge the profession of all loose practices and slovenly methods which may lower it in public esteem or hinder it from becoming a stronger force in the social order.

The establishment of a court or "contract committee," as suggested by Mr. Smith, to pass upon disputed questions would be an ideal way in which to establish such customs. That is precisely the

Mr. Himes. way in which the common law was originally developed, and is an eminently proper way in which to develop a common law that may be applicable to modern conditions. That such a court would have no legal status, is not material, for there is a large class of cases in which the parties only wish to be assured that they are treated fairly and according to custom. It is the suspicion of unfair dealing that breeds the larger number of disputes, and the decisions of such a court would be more satisfactory in their adjustment than the rulings of a court at law.

Although the decisions of the court would have only the force conferred upon it by the applicants, in time there would be established a series of precedents which could not be overthrown by the law itself.

Such a procedure is not without example, other than the original development of the common law. The British and North German Lloyds had a similar origin, in the association of vessel owners for the protection of their mutual interests, and the greatness of their success exceeds the wildest flight of the imagination.

An example of the work which such a committee might hope to accomplish may be found in the engineering custom of paying only for materials actually used in the work. The custom in vogue among builders and architects of measuring quantities according to various arbitrary rules is chaotic and illogical, and is distinctly inferior to the engineering method.

Mr. Aiken has mentioned some of the sophistries used by manufacturers in excusing their products. Such methods, to one who understands them, appear to be silly, but they cause lots of trouble among people who are unfamiliar with the business. Only an experienced man knows the value of experience, and it can hardly be expected that a dry-goods merchant will be interested in the phosphorus content of the steel for his new store building, or that a chief engineer who never saw a steel mill will take much interest in rail inspection.

Mr. Haring has related some of his troubles, and, perhaps, he at least will excuse the writer for referring to one of his own. There were large quantities of slope wall to build on three divisions, over one of which he had nominal control. The specification was impossible of execution, and he issued instructions to his assistants explaining what should be required. For this he was severely criticised, it being held that any such "let up" would afford an excuse for ignoring the specification entirely. After the work had been in progress about six months, his chief took him to task for exacting such good work of the contractors, saying that the men in charge of the other divisions were not so particular. It is plain that, by using a little reason in the first case, good work was obtained, whereas, on

the other divisions, in attempting the impossible, the result was a failure, and the work was as bad as it well could be. The solicitude of the chief for the welfare of the contractors was the cause of a change of management. Mr. Himes.

The opinion of Mr. Lovell, on the use of a blanket clause in the specifications, is commended to the profession. A man who does not know or cannot describe what he wants should get some one else to describe it for him.

Mr. Thompson emphasizes the need of uniformity of practice. This seems to be one of the aims of the United States Reclamation Service, and the writer has felt the need of it in his experience. The American Railway Engineering and Maintenance of Way Association is doing some excellent work in this direction, and its influence will be strongly felt in the profession.

Mr. Beahan's discussion has made very clear the meaning of a contract and the correct attitude of the engineer. His words should be studied with care. In them the young engineer may find more of value than in a whole library of technical books.

To Mr. Bixby, the writer is especially indebted for his kindness in giving to the Society some of his experience as an attorney in cases involving engineering work. His estimate of the value of a diary showing the progress of a piece of construction should be impressed upon every mind. He has also made it clear that an engineer is not a judicial officer, but that the value of his decisions rests on his expert knowledge and in his faithful obedience to the law which requires that his decisions shall be reasonable. In other words, his authority exists almost entirely because of his knowledge and integrity. If he is lacking in these, his decisions will not stand.

In summing up the discussion, the conclusion that seems to be most strongly impressed upon the mind is that, after having studied mathematics and the applied sciences, after having learned to run a transit and become an expert workman, if a man would attain eminence as an engineer, he must descend to the level of a newsboy and study human nature. He must begin at the bottom and seek to understand the ways of men, as individuals and as social units. He must acquire a knowledge of the laws of society and adjust himself so that he may move and act in harmony with its highest aims. Until he does this he is like a pebble in a pot-hole forced around and around by the rushing torrent. When he finally discovers his true position, when he discerns his correct relations to other beings and his surroundings, then, like an atom in Nature's chaos, he may select his affinities, unite in harmony with others and crystallize into a bulwark of society against which the tide may surge and the storms may beat, but over which they will not prevail.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1019.

TEST OF A THREE-STAGE, DIRECT-CONNECTED, CENTRIFUGAL PUMPING UNIT.*

By Philip E. Harroun, M. Am. Soc. C. E.

With Discussion by Messrs. J. Richards, C. D. Marx, Arthur L. Adams, Joseph N. Le Conte, Elmo G. Harris, W. B. Gregory, H. F. Dunham, Clyde Potts, Edwin Duryea, Jr., and Philip E. Harroun.

These notes are presented in the hope that they will lead to discussion and bring out the results of other tests.

For several years many of the makers of centrifugal pumps on the Pacific Coast have been claiming that they secure an efficiency of 75 and 80%, and even higher, for their pumps, but the writer has never been able to obtain authentic data which would support these claims. In fact, he has thought that an efficiency of from 55 to 60% was seldom exceeded, and that the average efficiency of the commercial pumps turned out was far less.

During the autumn of 1904, the writer, while on a trip through the San Joaquin Valley, had the opportunity of seeing in operation some eight or ten centrifugal pumping units, and, with the available instruments at hand, endeavored to obtain their efficiency. These were all two-step pumps, with one exception, which was a three-step. They were all direct-connected to induction motors. With two exceptions, they were being used for water-supply service

* Presented at the meeting of February 7th, 1906.

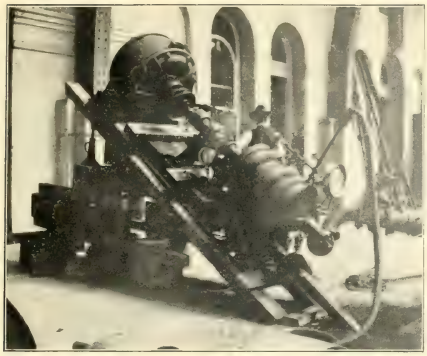


FIG. 1.—PUMPING UNIT SET UP FOR TESTING.

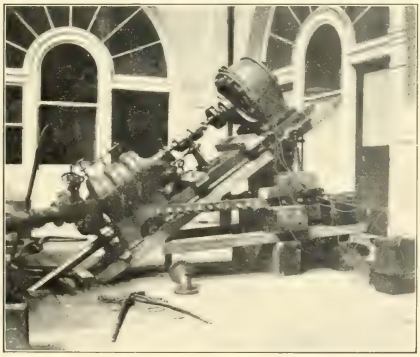


FIG. 2.—PUMPING UNIT SET UP FOR TESTING.

FIG. 3.—WEIR USED IN MEASURING DISCHARGE OF PUMP.

to the various towns in which they were located, and were pumping to stand-pipes or tanks a constant volume under a practically constant head, and were in continuous service. The two exceptions were irrigating units, under the same conditions as to volume and head, but in service probably an average of 6 months in the year. As, in these instances, the service was practically continuous, and both the volume and head were constant, these units were probably selected from stock, or designed to give a maximum efficiency for the volume and under the head in each particular case. The observed efficiencies of these units ranged between 25 and 45%, with an average of about 35 per cent. The writer had some hesitation in accepting fully the individual results, owing to his having no facilities for verifying the instruments used, and the necessity of accepting the statements of the owners as to certain essential factors; at the same time, he believes that the results he obtained are practically correct. In view of these facts, the writer was especially interested when the opportunity arose to test the unit mentioned in the following.

The unit consists of a three-stage, centrifugal pump, direct-connected to a 25-h. p., 2-phase, 7 200 alternations, 200-volt induction motor, making 1 120 rev. per min. It was built for use as a mine drainage or sinking pump, and to operate in an inclined shaft. The photographs, Plate VIII, show the unit in detail as it was set up for testing, and also indicate the approximate angle of operation. The contract for furnishing this pump simply provided that the pump should deliver 250 gal. per min. at a head of 200 ft., and that a standard 25-h. p. motor, as above described, would furnish all the power required.

Upon being retained by the purchasers to test the unit, the writer examined it in the place where it had been set up and was being operated by the makers. This examination showed that the unit was apparently delivering 240 gal. per min. at a head of 180 ft., and the failure of the unit to meet the prescribed requirements was stated by the makers to be lack of speed in the motor due to the excessive transforming of the current used. Under these circumstances, the writer refused to conduct the test until the requirements specified should have been more nearly approximated.

The writer also found that the facilities for testing offered by

the makers were not adequate, and he then sought permission to conduct the test at the Mechanical Laboratory of the University of California. This permission was readily granted by the Departments of Agriculture and of Mechanics, and, after considerable objection on the part of the makers, their sanction was also obtained.

Before the pump was shipped to the laboratory for the test, it was retained by the makers some two weeks, and during that time changes were made by them in the pump, or motor, or both, to bring them up to the requirements of the contract. Immediately prior to its shipment it was examined by the writer while in operation by the makers, and, according to their measurements, not verified by the writer, it was apparently fulfilling the requirements of the contract.

At the laboratory the unit was set up and run during the test by a representative of the makers. The control of the Departments of Agriculture and of Mechanics was exercised by Professor J. N. Le Conte, assisted by Mr. C. F. Gilcrest, in electricity; while the same services were exercised by the writer on behalf of his clients, the purchasers. Arthur L. Adams, M. Am. Soc. C. E., Consulting Hydraulic Engineer, was also present throughout the entire test.

It is needless to say that all possible precautions were taken to insure the correctness of the results obtained, and that there can be no question on that score. For the measurement of power, Weston watt meters were used. In the determination of the discharge, a 2.50-ft. contracted rectangular weir was used, and the results were calculated by the Francis formula, $Q = C\ (b - 0.2\ h)\ h^{\frac{3}{2}}$, using for C a coefficient determined by Professor Le Conte for this particular weir, which gives slightly greater results than the Francis coefficient. The weir discharge was also checked by volumetric tank measurements, and with an agreement of within 1 per cent.

Table 1 gives the results obtained, and the diagram, Fig. 1, covers the same items. At the head for which the pump was designed, 200 ft., the discharge was 190 gal. per min., and the combined efficiency was 34.2%; or, taking the motor at 90%, 38% was the efficiency for the pump.

It is very probable that the maker of this pump considered it his best effort to meet the conditions imposed in the contract. This is

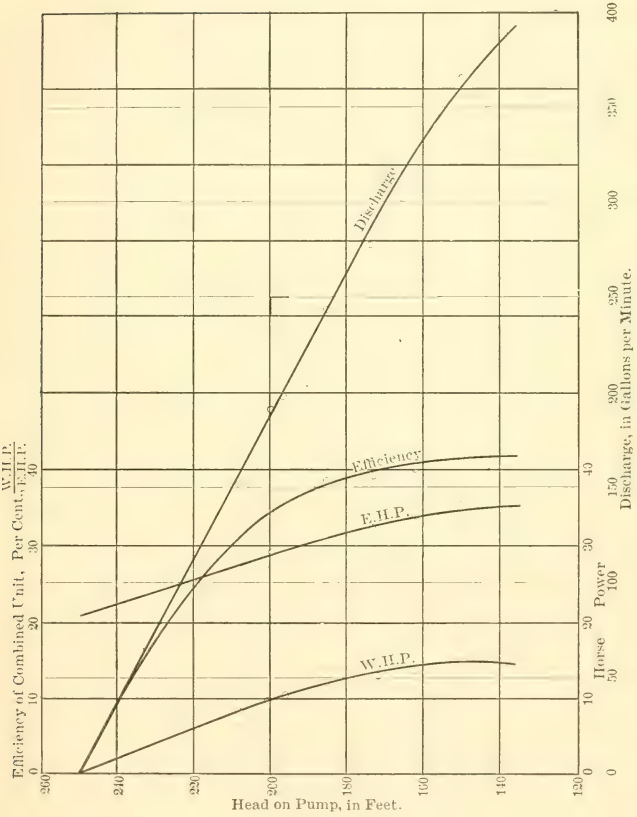

Efficiency of Combined Unit, Per Cent., W.H.P./E.H.P.
0
10
20
30
40
Discharge
Efficiency
E.H.P.
W.H.P.
Horse Power
0
10
20
30
40
Discharge, in Gallons per Minute.
0
50
100
150
200
250
300
350
400
260
240
220
200
180
160
140
120
Head on Pump, in Feet.

FIG. 1.

TABLE 1.—Test of Three-Step, Centrifugal Pump, with 4-in. Suction and 4-in. Discharge.

Test No.	Speed.	Watt Meters.		Kilowatts.	E. H. P.	Weir reading.	Weir zero.	h.	Gallons per minute.	Pressure Gauge.		Vacuum.		Volts.		Amperes.		Efficiency, combined unit.
		A.	B.							Pounds.	True.	Inches.	True.	A.	B.	A.	B.	
1	1 086	13.25	12.70	25.95	34.8	1.2365	1.016	0.2205	390	60	60	9	10	205	198	71	76	42.5%
2	1 110	13.5	12.9	26.4	35.4	1.237	1.016	0.211	366	65	65	8	9	208	201	71	75	41.7%
3	1 099	13.0	12.4	25.4	34.0	1.2075	1.016	0.1915	317	70	70	7	8	204	196	72	70	40.2%
4	1 103	12.3	11.9	24.2	32.4	1.1915	1.016	0.1755	279	75	75	6	7	204	194	71	69	39.4%
5	1 112	11.65	11.3	22.95	30.8	1.176	1.016	0.160	244	80	80	6	7	207	200	65	63	38.6%
6	1 112	10.65	10.5	21.15	28.3	1.155	1.016	0.139	198	85	85	5	6	206	198	60	59	35.9%
7	1 135	9.95	9.8	19.75	26.5	1.132	1.016	0.116	152	90	90	5	6	204	198	57	55	31.2%
8	1 146	9.4	9.3	18.7	25.1	1.109	1.016	0.093	111	95	95	5	6	205	199	54	52	25.2%
9	1 126 to 1 131	10.8	10.5	21.3	28.55	1.152	1.016	0.136	190	87	87	5	6	203	197	62	50	34.2%
		10.9	10.7	21.6	28.95	1.152	1.016	0.136	190	87	87	5	6	.....	.....	.....	.....	
		10.9	10.7	21.6	28.95	1.152	1.016	0.136	190	87	87	5	6	.....	.....	.....	.....	
		10.9	10.7	21.6	28.95	1.152	1.016	0.136	190	87	87	5	6	.....	.....	.....	.....	
		11.1	10.85	21.95	29.4	1.152	1.016	0.136	190	87	87	5	6	.....	.....	.....	.....	
		11.2	10.9	22.1	29.6	1.152	1.016	0.136	190	87	87	5	6	.....	.....	.....	.....	
	Mean	.....	.....	.....	29.07	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	
Gate shut tight....		7.6	7.7	15.3	20.5	0	.....	0	0	.....	.....	.....	.....	.....	.....	.....	.....	
Water drained out.		0.6	1.3	1.9	2.55	0	.....	0	0	.....	.....	.....	.....	.....	.....	.....	.....	

Note.—Tank Measure.
Rise in Level.
167.00

Time. Q, in second-feet.
10 min. 0.424.
0.424 sec-ft. = 190 gall. per min.

indicated by the changes made in it during the two weeks it was held by him prior to the test, and also by the probability that he would not have submitted it for such test had he deemed it otherwise. That it fell so far short of the maker's expectations, is due, to some extent, to the crudeness of the apparatus by which the shop tests are made. It is much to be regretted that the test was not conducted through a greater range, but it seems probable that this pump should be rated at 400 gal. per min. at 130 ft. head, and, allowing 90% efficiency for the motor, 47% would be the maximum efficiency of the pump at this rating.

This test is specially interesting to the writer, in view of the approximate tests previously mentioned, and, in connection with them, he believes that 35% may be taken as a fair average of the efficiencies obtained in the actual operation of these units in this section.

It seems reasonable to expect that, for so small a unit as that tested, a pump efficiency of from 55 to 60% should have been obtained, or, say, 50 to 54% for the combined unit. If $50 per horsepower per annum is taken as the cost of power, the annual charge for the unit tested is $1 450, as against $1 000 for a unit of 50% efficiency, and the annual saving in cost of power, by a unit of 50% efficiency, would have returned the entire first cost of the one tested in 2½ years.

DISCUSSION.

Mr. Richards. J. RICHARDS, ESQ. (by letter).—The experiments described by Mr. Harroun were conducted at considerable expense and effort to educate engineering students in the University of California, but they might have served a more useful purpose had they been directed toward solving some of the present problems in centrifugal pumping, instead of testing the efficiency of a commercial pump, of which not even the construction is disclosed, and which, for that reason, cannot be discussed on technical grounds.

The writer feels warranted in this remark because in August, 1904, he proposed to send to the faculty of the University of California several pumps, not made for sale, but for experimental purposes. These pumps were arranged with adjustable and removable dispersion vanes, with throatways finished smooth, and adapted for a number of different experiments, including relative efficiencies.

The experiments proposed were as follows:

1.—3-in. pump. With open throat, guide vanes removed, tested at various heads and rates of speed. Cut off points of the balancing vanes on the back until the impeller is in equilibrium, and no thrust.

2.—3-in. pump. With guide vanes set for $\frac{1}{4}$-in. throats.

3.—3-in. pump. With guide vanes set for $\frac{1}{2}$-in. throats.

4.—Remove outside vanes and open impeller at the sides; then repeat Experiments 1, 2, and 3.

5.—Connect 4-in. discharge to this pump, remove dispersion vanes, and test for heads from 10 to 50 ft.

Experiments 6, 7, and 8 to be defined later.

9.—2-in. pump. With dispersion vanes removed. Cut off back vanes on this impeller to balance, as in the 3-in. pump.

10.—2-in. pump. With dispersion vanes for $\frac{1}{4}$-in. throatways.

11.—2-in. pump. With dispersion vanes for $\frac{1}{2}$-in. throatways.

12.—Second 2-in. pump. With single issue (Pump No. 3.) This is a novel combination, now the subject of controversy among engineers.

13.—3-in. four-stage pump. Eight experiments, with one, two, three, and four impellers, with and without dispersion vanes, at pressures from 10 to 150 lb. per sq. in., and speeds accordingly.

At that time the machinery was almost completed, and the writer stated, in his letter to the University of California, that no complete experiments had been made which had offered material aid to the art; that highly useful experiments had been made in France and Switzerland, but were mainly in private interest; that it was not intended, in this case, to try a certain construction of commercial

pumps, but that the machinery, except in Experiment 13, had been made especially for the purpose and for no commercial object whatever. Mr. Richards.

To this communication the writer received no response.

In respect to the performance of stage centrifugal pumps, the writer, for some years past, has not given much attention to the claims of those who make and sell such machines. Usually, the means of determining the useful effect, or efficiency, are incomplete, or the results inconclusive.

Some years ago, the writer sent to a celebrated engineer in Switzerland a table of quantities and results prepared by the makers of a compound centrifugal pump, that is, two pumps in series. This table was prepared, no doubt, in entire good faith, but the laconic reply received from the Swiss engineer was discouraging.

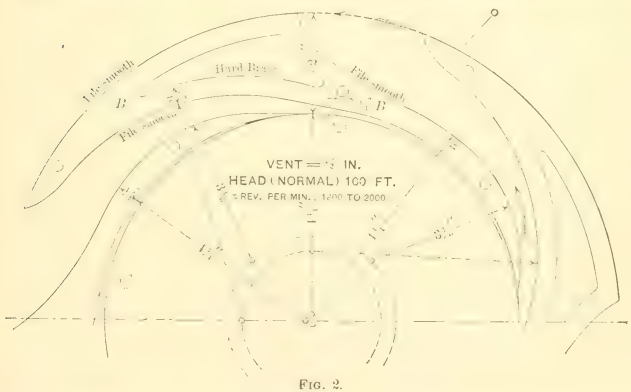


FIG. 2.

In substance, he said, "These quantities will not in this country produce the result claimed, and if they do in California it must be owing to the climate."

Another case may be mentioned. Two small pumps were purchased from different makers to operate against a head of 100 ft., and were guaranteed to operate at an efficiency of 70 per cent.

The pumps were put into the hands of a careful engineer engaged in such tests, and the efficiency attained at the head named was nearly identical with that given by Mr. Harroun, 34 per cent. Table 2 gives the results of the tests by Mr. Benjamin, of New York.

These facts are mentioned to support the remark made concerning statements of the efficiencies attained by common, stage

Mr. Richards. centrifugal pumps, with their operating surfaces rough, and water ducts of unfinished castings.

TABLE 2.—TEST OF 2½-INCH CENTRIFUGAL PUMP.

Revolutions per minute.	Total pressure.	Gallons per minute.	Electrical horse-power.	Water horse-power.	Wire efficiency.	Approximate pump efficiency.
1 520	31	204	11.7	3.68	31.4	40.2
1 568	32	182	10.85	3.39	31.3	40.3
1 566	36	146	10.72	3.06	28.6	37.1
1 591	34.5	113.3	9.78	2.54	26.0	34 3
1 549	39	106.5	9.67	2.42	25.0	33.2
1 560	39	100.0	9.64	2.27	23.6	31.5
1 557	39	83.7	9.00	1.9	21.1	28.4
1 568	41	43.5	7.92	1.04	13.15	18.1
1 562	42.5	26.1	7.6	0.645	8.5	11.72
1 610	45	0	7.03	0	.....	.....

TEST OF 2-INCH COMPOUND CENTRIFUGAL PUMP.

1 192	51.25	112	13.27	3.34	25.2	31.5
1 217	52.75	108.3	13.38	3.33	24.9	31.0

Centrifugal pumping, although only a little more than half a century old, is fast falling into a scientific form and a standard construction, modified, of course, by the circumstances of use. There seem to be left, in fact, but two constructive features and two phenomena of operation with which to deal.

The constructive points are, means to conserve the kinetic energy of the water's revolution, or its tangential energy, as it is sometimes called; and to produce an equilibrium of the impellers to avoid lateral thrust.

The phenomena are, the effect produced, on a stream of water moving at high velocity, by roughness or irregularity of surfaces; and the kinetic stability or tendency of a body of water in rapid revolution to remain in one plane.

Besides these features and phenomena there are the conditions of adaptation and endurance, which, in a sense, are determinate, and are not problems, but matters of skill and engineering knowledge.

It cannot be expected, nor is it possible, to discuss at present the features and phenomena mentioned, as it would require diagrams and other means of illustration not at hand.

The writer constructed on the Pacific Coast, and successfully applied in actual use, the first, stage centrifugal pumps, of which he has any knowledge. At his own expense, he has made two trips to

Mr. Richards.

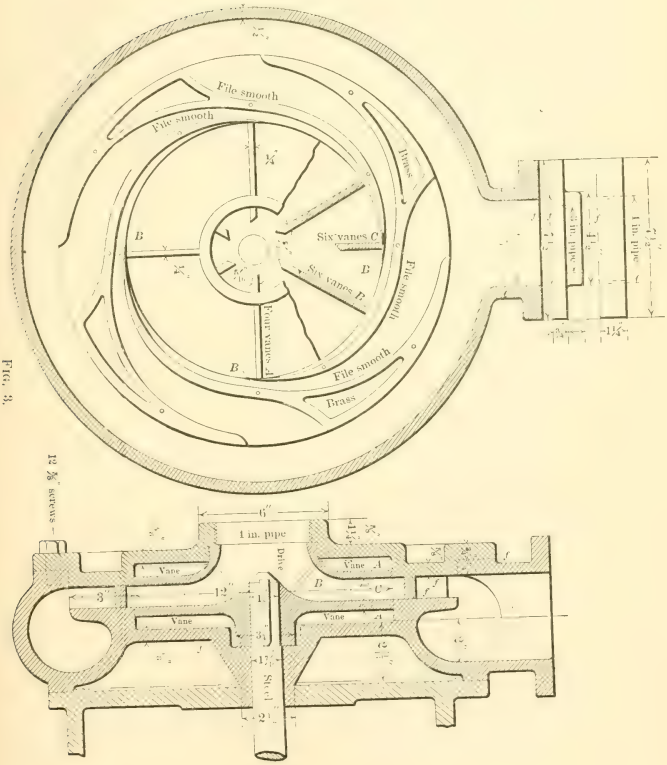


FIG. 3.

Mr. Richards.

Europe especially to investigate the progress of the art there; has contributed a great deal in effort and money to improve centrifugal pumps, and thinks he may fairly claim to have added something to the art. The emoluments have been inconsiderable, and he is not now engaged in or interested in the manufacture.

The practice on the Pacific Coast, while early and extensive, cannot at the present time be called advanced. The pumps required, being mainly for irrigation and rough uses, are good enough, perhaps. They are, to a great extent, bought and used for a few months of the year, and stand neglected for the remainder of the time. Such things are determined by natural adaptation, and must be so judged.

A centrifugal pump, finished in its operating zone, or provided with dispersion vanes, must cost at least twice as much as one without these features, and the added efficiency may or may not warrant the more expensive construction.

It is strange that little or no investigation of the subject has been made in the numerous scientific laboratories of our day. The lateral thrust on impellers was the subject of some very interesting experiments made at the University of California in 1887 and published in a bulletin entitled "A Hydraulic Step." Other experiments, so far as known, have been made, as in the present case, merely to determine the effect of commercial machines, and consequently are of no especial value. In other words, they affect only a particular construction.

The machines offered to the university are of a purely experimental nature, admitting of a dozen or more modifications. They are now stored in San Anselmo, waiting a time when the writer can make the proposed experiments. The machinery also includes apparatus to determine the practicability of combining air and water, subjecting this combined fluid to centrifugal force, and then separating the air at the pressure due to the centrifugal effect.

The importance of this last-named operation will be at once perceived. The suggestion of such a process came from a well-known engineer of San Francisco, and the apparatus was constructed nearly two years ago, but not set in operation except to determine its capacity for speed and endurance.

The apparatus now at the University of California, together with what the writer proposed to furnish, would have constituted a very extensive outfit, and provided for much useful investigation.

This brief discussion will be closed with a remark on the "Shibboleth" of "efficiency," as applied to centrifugal pumps. Efficiency is but one of many conditions to be considered. Adaptation and endurance are equally essential. A centrifugal pump, to operate at a high efficiency, is necessarily limited to special purposes. With

refined machines, 80% of useful effect is attainable, but one that will utilize 60% of the power applied is likely to be the best machine to buy. The efficiency attainable in various uses is from 40 to 80%, being lowest in dredging machines, but the strange feature of the matter is the constant inquiry respecting efficiency. A customer will go to a maker or dealer in displacement pumps and purchase one to operate at an efficiency of 25 to 35% and never make an inquiry or ask a guaranty of effect. Indeed, the word "efficiency" is not found in the trade circulars relating to direct-acting piston pumps. Then why should it be so constantly a test for centrifugal pumps? Mr. Richards.

C. D. Marx, M. Am. Soc. C. E. (by letter).—This paper brings out the fact clearly that centrifugal pump manufacturers (and, in that respect, they do not differ much from other manufacturers) are in the habit of guaranteeing efficiencies for their pumps which they cannot reach in an actual test. Sometimes the purchaser is satisfied with the guaranty and the name of the manufacturer, and makes no test. This method is always very satisfactory to the manufacturer. Sometimes, however, the purchaser is foolish enough to employ an engineer, who proceeds to find out if he is getting, for his employer, what the manufacturer has agreed to furnish. In many cases there will be disappointment on both sides. The manufacturer will find that, whether unintentionally or not, he has promised more than he can fulfill; and the purchaser will probably find himself compelled to adopt an inefficient plant, because of the delay and trouble incident to change from one type of construction to another. Prof. Marx.

The writer is not stating an imaginary case. A couple of years ago, bids were invited for the enlargement of a municipal waterworks in California. The engineer in charge of construction, in his call for bids and in his specifications, merely stated the requirements to be filled, and left the individual bidder free to offer his own solution of the problem. Among other requirements was one calling for the pumping of from 400 to 800 gal. per min. against a head ranging from 116 to 232 ft.

One of the leading manufacturers of centrifugal pumps on the Pacific Coast guaranteed to furnish a single centrifugal pump capable of doing this work with a pump efficiency of 65 per cent. The original contract for this work specified one pump fitted with two runners so as to obtain two discharges, 400 and 800 gal. per min., with practically the same efficiency. The following is quoted from the engineer's report to the town officials:

"The firm, however, was somewhat hasty in making this guaranty, for when they came to making the design they found that they could not meet the requirements. In lieu of this they offered to supply us with two pumps, one to operate between 400 and 600 gal. per min., and one between 600 and 800 gal. per min., with an

Prof. Marx. efficiency at the minimum and maximum ratings of 65 per cent. This construction would be somewhat more convenient to us, on account of making changes when necessary, and I agreed to the change. One of these pumps was made, but when tested it was found to give not more than 50% efficiency, so that the firm is now fitting the pump with new bronze runners polished inside and out."

A test of this pump gave an efficiency (ratio of water horse-power to horse-power delivered at the pump shaft) of 62.9% when pumping 603 gal. per min. against a total head of 110 ft.; and 57.3% efficiency when pumping 437 gal. per min. against a head of 116 ft. This same pump, pumping 425 gal. per min. against a head of 220 ft., gave an efficiency of 49.4%, and, when delivering 620 gal. per min. against a head of 217 ft., the efficiency was 57.5 per cent.

As the result of this test, the manufacturer submitted a supplemental agreement which he wanted the town authorities to sign. In this occur the following phrases:

"Also because the contractor found it difficult to meet the contract efficiency of 65% in a single pump for the large range of capacity of 400 to 800 gal. per min., and also the large range of head, 116 to 232 ft., when making all deductions under the rulings of the engineer in charge.

"It being further conceded by both parties to this agreement that it is a difficult matter to make one centrifugal pump meet these specifications when being driven by a belt from a line shaft of only one speed, to wit, 300 rev. per min.

"But at the same time, both parties to this contract believe that a centrifugal pump is best adapted to this service when the exact quantity of water and head to be pumped against are fully determined."

The inference, from the last clause, naturally is that the engineer did not know the quantity of water to be pumped, or the head.

Since both head and quantity are likely to vary within wide limits in a municipal supply, the conditions were stated in the specifications. The pump manufacturer was familiar with them, gave a guaranty to fulfill them and failed to do so.

Mr. Adams. Arthur L. Adams, M. Am. Soc. C. E. (by letter).—Mr. Richards, in his excellent discussion of Mr. Harroun's paper, questions the value to the profession of such a test as that described, because it attempts no analysis of the causes leading to the failure of the unit to attain a high efficiency or to meet the requirements intended by both its purchasers and manufacturers. The writer, on the contrary, considers that all such carefully conducted tests are of great value.

To the mechanical engineer, highly specialized in the design of centrifugal pumps, they contribute little, but to the engineer in more general lines of practice, who is frequently called upon to decide as to the type of pump suited to meet best the requirements of conditions not often identical in different cases, it is of the

greatest interest and value to know what degree of efficiency one may reasonably expect to secure from the pumps of the better known manufacturers, constructed with full knowledge of the conditions under which they are expected to work. Mr. Adams

Exact information of the character set forth in the paper is the more desirable:

First.—Because of the broad claims of high efficiency—70 to 80% for the pump alone—which are put forth by the manufacturers of San Francisco;

Second.—Because but a small proportion of installations are ever actually tested after their completion, and very few reliably conducted tests are of record;

Third.—Because it is a fact, known to many engineers coming into contact with such installations in California, of which there are a great many, that it is most exceptional for efficiencies approaching the before-mentioned claims of the manufacturers to be attained under ordinarily good working conditions.

The centrifugal pump is splendidly adapted to meeting requirements in certain fields of service. Those fields are doubtless being extended legitimately in some quarters, under the skilful designing of the best specialists, but the state of the art in California, as it is being applied in the manufacture of these pumps, is a question of great importance, that engineers may be able to restrict their use to the province of their real merit; and concerning this the paper adds to our knowledge.

Joseph N. Le Conte, Esq. (by letter).—In reference to Mr. Richards' communication to the University of California, the writer desires to state that, to the best of his knowledge, such a letter was never received by any one now connected with the university. If such an offer were made, the apparatus would be gladly received, and any tests that Mr. Richards might desire would be made. The Department of Hydraulics is always most grateful for any additions to the laboratory, and there is no doubt whatever that the experiments mentioned can be made, as the results of the tests on Pelton wheels will show. Prof. Le Conte.

Elmo G. Harris, M. Am. Soc. C. E. (by letter).—It is a matter to be regretted that, under the pressure of severe competition, manufacturers of centrifugal pumps claim, and will often guarantee greater efficiencies than their machines can give; but there is no hope of bringing about the desired reform except by enforcing these guaranties. If this were commonly done, it would result in a twofold benefit: first, by preventing a mild degree of fraud on credulous purchasers; and, second, by ultimately improving the machines, until the efficiency is brought up to what the well informed believe to be economically attainable. Mr. Harris.

Mr. Harris. If the equivalent of the following clause were inserted in contracts for the purchase and installation of centrifugal pumps, the makers would be very conservative in their guaranties of efficiency, and, further, it would reveal to all parties concerned an economic principle too often neglected in fixing the cost of various installations:

"The contractor guarantees an efficiency of ( ) per cent., as determined by dividing the power delivered to the pump shaft by the power (due to the pump) in the water taken immediately after leaving that portion of the plant for which the contractor is responsible; and should this efficiency not be realized, the deficit, under normal working conditions, shall be estimated in horse powers, and, assuming the value of 1 h. p. to be ($) per annum, the present value of such a sum, paid annually for ( ) years, shall be estimated under a rate of interest of ( ) per cent. per annum, and this sum shall be deducted from the contract price of the pumping plant * * *."

If a corresponding bonus should be offered for exceeding a stated efficiency, the rapid improvement of centrifugal pumps would be assured. Much of the fault is due to the indifference or ignorance of the purchaser.

In regard to needed experimental knowledge, little is known about the losses caused by the friction of water gliding over metallic surfaces at such velocities as exist in centrifugal pumps. This should be found for revolving disks varying in diameter, surface finish, and speed. Considering the value of such data, the cost of the necessary apparatus would be small.

The answer to Mr. Richards' question, as to why efficiency should be insisted upon in centrifugal pumps and not in direct-acting reciprocating pumps, is that the cost of the power going into a centrifugal pump is very much greater than that going into a direct-acting pump; the latter is a cheap affair in first cost, taking steam direct from the boiler, while for a centrifugal pump there must be a rotating engine of some sort, and, if the pump is driven by electricity, there must be the prime motor (steam engine or water-wheel), the dynamo and the electric motor.

Noting Mr. Richards' allusion to a scheme for compressing air by combining air and water in a centrifugal machine, the writer would call attention to the Appendix* to the paper entitled, "Theory of Centrifugal Pumps and Fans," where such a scheme is somewhat clearly outlined.

Professor Le Conte was probably too modest to mention his excellent report of tests of centrifugal pumping plants published in the United States Report of Irrigation and Drainage Investigations, 1904. Such matter is scarce and valuable.

* *Transactions*, Am. Soc. C. E., Vol. LI, p. 222.

Mr. Gregory.

W. B. Gregory, Esq.* (by letter).—The writer has not had any experience with three-stage pumps, but, during the summer of 1905, he tested a two-stage, direct-connected, centrifugal pumping unit. This pump had 6-in. suction and 4-in. discharge pipes, and was driven by a direct-current motor on the same bed-plate. The discharge head was measured with a pressure gauge on the discharge pipe near the pump. The suction was measured with a vacuum gauge on the suction pipe near the pump; both gauges being carefully calibrated.

The total head was ascertained by reducing these observations to feet of water, correcting for the difference of level, and adding to the difference between the velocity heads in the discharge and suction pipes. Thus the pump is given credit for the velocity head it produces, as well as for pumping the water against pressures equivalent to the given heads.

The water discharged from the pump was measured by an 18-in. Cippoletti weir, placed in a tank with baffle plates arranged so that the water flowed quietly to the weir. The depth of the water over the crest of the weir was measured by an accurate hook-gauge.

The electrical readings were obtained from carefully calibrated instruments. The electrical losses were measured and corrections made.

The friction in the pump and motor was obtained when the pump was not primed, and, in getting the efficiency of the pump, one-half was charged to each, assuming the friction to be constant for all loads.

TABLE 3.—Two-Stage Turbine Pump.

Time.	Revolutions per minute.	Volts.	Amperes.	Electrical horse-power.	Suction, in feet of water.	Discharge, in feet of water.	Total head, in feet of water.	Depth over weir, in feet.	Discharge, in cubic feet per second.	Water horse-power.	Efficiency of pump and motor: percentage.	Efficiency of pump: percentage.	Efficiency of motor: percentage.
9:50...	1 000	140	101	18.95	2.20	136.2	142.8	0	0	0	0	0	0
9:55...	978	138	126	23.33	3.06	140.0	147.6	0.241	0.597	9.98	42.8	50.2	85.3
10:00...	967	136	176	32.11	6.46	127.3	139.1	0.322	0.923	14.52	45.2	51.9	87.1
10:03...	955	134.3	194	34.95	8.72	115.7	130.3	0.362	1.101	16.25	46.5	53.1	87.6
10:06...	960	134	208	37.38	10.20	104.2	120.7	0.378	1.216	16.62	44.4	50.6	87.7
10:09...	960	134	217.3	39.05	12.23	92.5	111.5	0.408	1.316	16.60	42.5	48.3	88.0
10:12...	956	133.5	225	40.26	13.48	81.0	101.6	0.427	1.409	16.20	40.2	45.7	88.1
10:15...	956	133.5	236	42.25	14.95	69.4	92.0	0.446	1.507	15.67	37.1	42.0	88.3
10:18...	956	133.5	242	43.32	16.20	57.8	82.0	0.459	1.571	14.56	33.6	38.1	88.3
10:21...	949	132.5	247.5	43.97	18.01	46.3	72.7	0.476	1.657	13.62	31.0	35.1	88.3
10:24...	949	132.5	254.2	45.17	19.37	34.7	62.9	0.489	1.727	12.30	27.2	30.8	88.4
10:27...	949	132.5	260.5	46.31	21.01	23.1	53.1	0.495	1.759	10.57	22.8	25.8	88.4
10:30...	949	132.3	264.0	46.88	21.53	16.2	46.7	0.495	1.759	9.30	19.8	22.4	88.4

* Irrigation Engineer, Office of Experiment Stations.

Mr. Gregory.

Observations were taken, beginning with the discharge valve closed, then opening it slightly, and again taking observations as soon as all the conditions were constant. This process was continued until the discharge valve was wide open.

Some trouble was experienced with the thrust-bearing on account of heating, but the test was not interfered with seriously.

The discussion of the efficiencies of centrifugal pumps in general is of great interest. The writer has had considerable experience in testing a great many varieties of these pumps during the last ten years. Before entering into a discussion of results, it will be well to define exactly what is meant by the term efficiency, which,

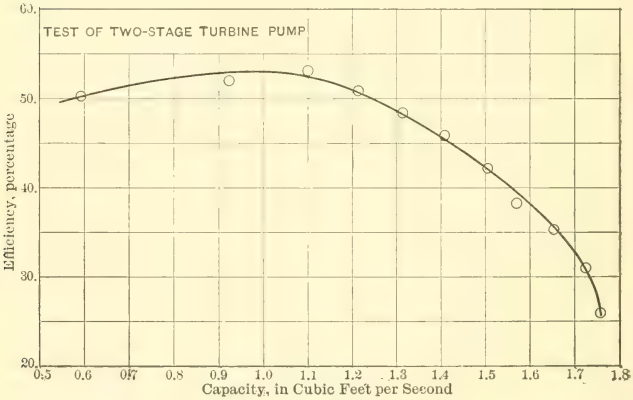


FIG. 4.

in general, is the ratio of the output of a pump to the energy furnished to the shaft. With pumps used for elevating water it seems to be fair to use, for the output of the pump, the useful work, or a certain number of pounds of water actually raised through a distance, measured in feet. If the first quantity is expressed in pounds per second, the product of pounds by feet of head, divided by 550 will give the horse-power corresponding to the useful work of the pumps. Usually, the water is discharged with appreciable velocity, and, if the total output of the pump, including the piping, is to be credited to the pump, the head equivalent to the velocity of discharge must be added to the height through which the water is elevated to obtain the total head. By either of the foregoing

methods, the loss at the entrance to the suction pipe, and the friction losses in the suction and discharge pipes are charged to the pump. Mr. Gregory.

In testing pumps in which the velocities are high, as in the case of hydraulic dredges, and in any case where the pump is to be eliminated from its special arrangement of piping, the total energy developed by the pump is desired, so that comparisons may be made of the pump performance only. In such cases the pressures are obtained near the pump on the suction and discharge pipes, and the

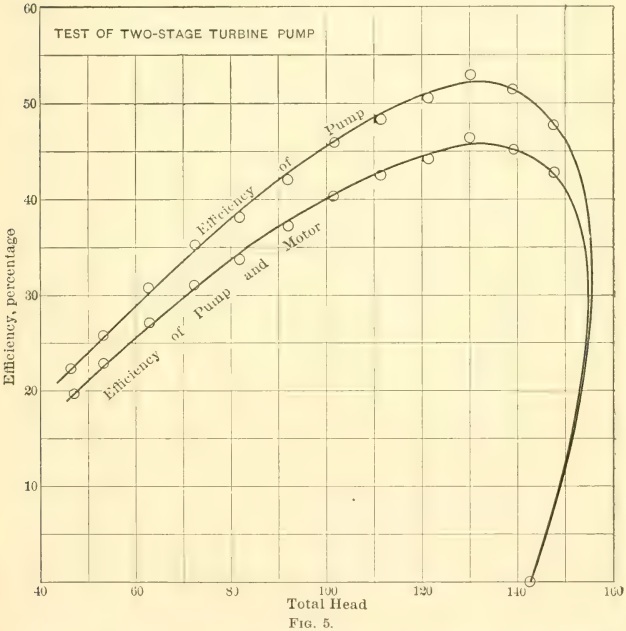


FIG. 5.

quantity of water discharged is measured. From these quantities the velocities in the suction and discharge pipes become known, and the total output of energy may be computed in the manner described in the test of the two-stage pump, as stated previously. When low velocities are used, the difference between the total energy and the useful work will often be small, particularly if the suction pipe is enlarged where the water enters it and the discharge pipe is

Mr. Gregory. enlarged where the water leaves that pipe. With the high velocities used in dredging, the difference between the total energy and the useful work is much greater; often the discharge level is practically that of the suction side, and the useful work consists in developing a high velocity and consequently great kinetic energy in order that material may be carried along with the water. It is evident, therefore, that, in the case of dredge pumps, the total-energy basis must be used.

It is customary to report tests of ordinary pumping plants on the basis of useful work, and tests of dredge pumps on the basis of the total-energy output. This should be kept in mind in comparing efficiencies.

The writer has tested large pumps, used for elevating water, which had efficiencies as high as 75% when based on useful work. Such pumps are not at all common; they are the exception and not the rule. From 60 to 65% represents more nearly the average case, and it is not at all difficult to find examples which fall much below these figures.

Tests of pumps used in hydraulic dredging were made by the Mississippi River Commission in 1902 and 1903. The total efficiencies were found to range from about 57 to 68% in the various cases. These results are very good, and will compare with those obtained from many centrifugal pumps using lower velocities. The pumps were large, and the diameters of the discharge pipes varied from 32 to 36 in.

Mr. Dunham. H. F. DUNHAM, M. AM. SOC. C. E.—If the speaker is not mistaken, the author has failed to mention the size of the centrifugal pump or the diameter of the rotor. Dimensions and capacity are so closely related to the percentage of efficiency obtainable that it would be well to include them when possible. The speaker's rather limited acquaintance with centrifugal pumps and their manufacture does not coincide very closely with the author's experience. The manufacturer usually knows pretty accurately, from his own shop tests or published tests, the efficiency of the pumps that he puts upon the market, and, while now and then an anxious salesman or a man not anticipating any check upon his statements may be extravagant to the degree indicated, yet the speaker believes that generally, in the East, it would be difficult to find a manufacturer who claimed 50% for a small centrifugal pump. When the capacity is much increased—perhaps to a No. 7 or No. 8, in which the number indicates the diameter of the discharge, and the working head is not very great—there might be some assurance from reliable manufacturers of an efficiency approaching 50 per cent. It would be of interest to know whether the maker of the pump in question first put it out with that guaranty of efficiency, or whether the guaranty was

made by some agent unacquainted with facts usually known to the builder, who was thus drawn into the discussion and led to "take chances" upon the result. The speaker recognizes a wide difference between ordinary oral "claims" for efficiency and the incorporation of such "claims" in the terms of a contract. Mr. Dunham.

CLYDE POTTS, ASSOC. M. AM. SOC. C. E. (by letter).—This paper and the discussion it has brought forth have added valuable information on which a designing engineer can draw when writing specifications for centrifugal pumping machinery. There is something radically wrong with specifications as they are now written, or with possible, should build pumps which will fulfil the terms of the specifications to fit the best pump on the market; or pump makers, if possible, should build pumps which will fulfil the terms of the specifications. It is embarrassing alike to maker, purchaser and engineer to have a pump fail to meet the specifications. Experimental data of this kind are always extremely valuable, for while such information may or may not aid the mechanical engineer in the design of pumps, it is an aid to the hydraulic engineer, as it shows him what can be expected from the centrifugal pumping unit in question. Mr. Potts.

All the pumps described have dealt with very high heads, and, to the data set forth, the writer wishes to add the results of some experiments which he made during the summer of 1905 with centrifugal pumps working against low heads. These pumps also differ, in that they pump sewage, whereas those described pumped clear water. The pumps tested are duplicate 8-in. centrifugal pumps of the ordinary type, with a suction of about 8 ft., and a discharge head varying from $11\frac{1}{2}$ to 37 ft. They are direct-connected to vertical, compound, condensing, high-speed engines. The engines are supplied by steam from boilers in an adjoining room, the boiler pressure being about 80 lb. Shortly after the engines were installed they were given an acceptance test. Under the specifications the plant would show a duty of 25 000 000 ft-lb. per 100 lb. of coal consumed. The plant was also required to pump a certain number of gallons against a given head. During the acceptance test the duty from the pumps was found to be as shown in Table 4.

TABLE 4.—DUTY, IN FOOT-POUNDS PER 100 LB. OF COAL.

Length of Run, in hours.	Duty.	Length of Run, in hours.	Duty.	Length of Run, in hours.	Duty.
5	23 180 000	12	24 160 000	18	23 960 000
7	24 121 000	13	24 280 000	19	23 741 000
8	23 960 000	14	24 264 000	20	23 805 000
9	24 400 000	15	24 100 000	21	23 086 000
10	24 480 000	16	24 232 000	23	23 712 000
11	24 000 000	17	24 040 000		

Mr. Potts. At no time, as will be seen, did the pumps fulfil the specifications for duty; on the other hand, they were of sufficient capacity to deliver, against the given head, one-fourth more sewage than required by the specifications. Under these considerations, the pumps were accepted and installed. This acceptance test was made in August, 1896, with clear water, while the pumps were yet new, and before any sewage had been pumped.

After the pumps had been running for nine years, the writer gave the plant a thorough test, to determine the present duty of the engines, and also to determine what quantity of sewage they would pump against the head then existing. These tests were necessary,

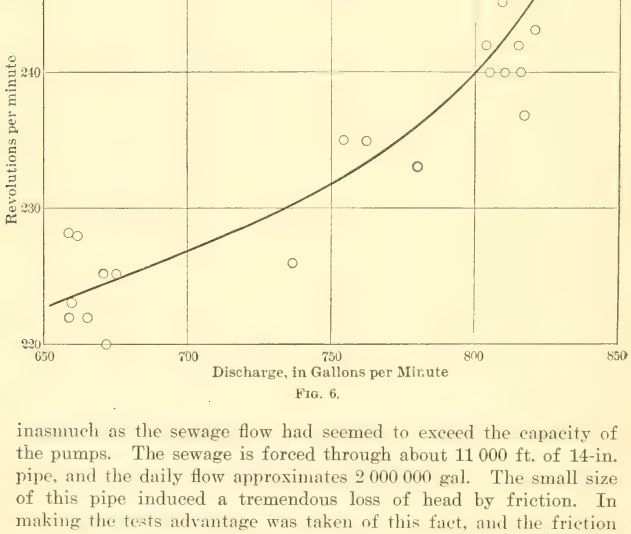


FIG. 6.

inasmuch as the sewage flow had seemed to exceed the capacity of the pumps. The sewage is forced through about 11 000 ft. of 14-in. pipe, and the daily flow approximates 2 000 000 gal. The small size of this pipe induced a tremendous loss of head by friction. In making the tests advantage was taken of this fact, and the friction head in the pipe was varied by a relief valve near the pumps, so as to give different heads for the tests. The flow was measured by the ordinary weir method as the sewage entered the pumping chamber. The pressure in the discharge was taken by a pressure gauge. A gauge also gave the suction head. The engines were equipped with indicators, cards were taken continuously during the tests,

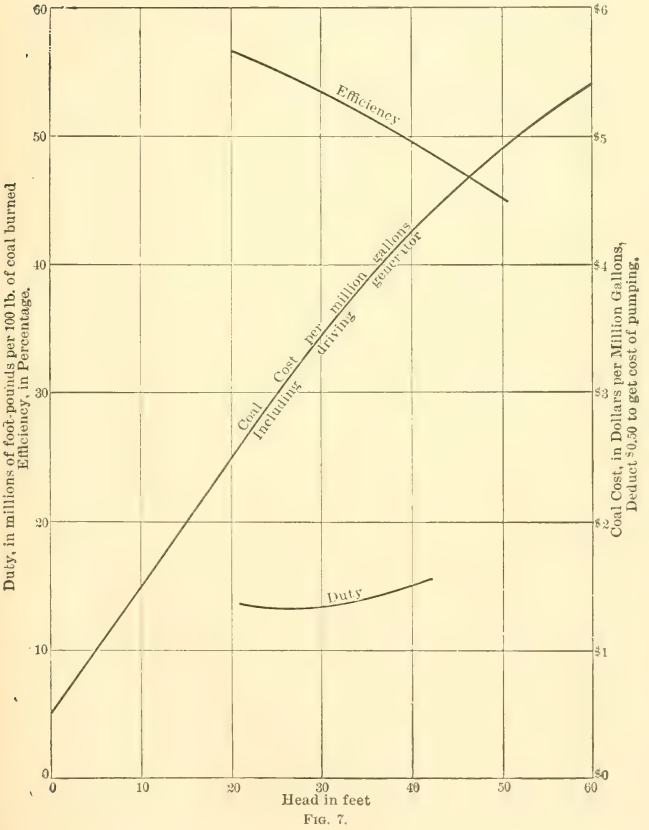

Duty, in millions of foot-pounds per 100 lb. of coal burned
Efficiency, in Percentage.
Coal Cost, in Dollars per Million Gallons,
Deduct $0.50 to get cost of pumping.
Head in feet
Efficiency
Coal Cost per million gallons driving generator including
Duty
0
10
20
30
40
50
60
$0
$1
$2
$3
$4
$5
$6

FIG. 7.

Mr. Potts. and careful measurements of the coal and feed water were made, this portion of the work being in charge of M. J. A. Wheeler, M. E. The coal used was buckwheat anthracite costing $3.19 per ton of 2 000 lb. in the bins. The results of these tests are shown in Table 5.

TABLE 5.

Run number.	Head, in feet.	Coal burned per hour, in pounds.	Discharge, in pounds per hour.	Foot-pounds per hour.	Duty, in foot-pounds per 100 lb. of coal.	Cost, per million gallons.
1.....	21.37	144.5	737 460	19 720 000	13 650 000	$2.06
2.....	28.87	185.0	716 400	24 650 000	13 330 000	2.82
3.....	41.97	216.0	712 800	33 800 000	15 650 000	3.88

Table 5 shows that the duty now obtained is only about 13 000 000 ft-lb. per 100 lb. of coal.

TABLE 6.—Relation Between Speed of Pumps and Discharge.

Speed, in revolutions per minute.	Discharge, in gallons per minute.	Speed, in revolutions per minute.	Discharge, in gallons per minute.	Speed, in revolutions per minute.	Discharge, in gallons per minute.
227	661	226	737	246	818
222	659	235	755	252	818
223	659	235	763	243	821
223	659	233	780	249	821
220	659	240	788	237	818
223	659	240	791	250	815
222	664	242	805	240	815
220	672	245	810	245	813
225	672	250	813	240	805
225	675	254	815	.....	.....

Table 6 shows the relation between the speed, in revolutions per minute, and the discharge, in gallons per minute. This relation is shown graphically in Fig. 6.

A study of Table 6 and Fig. 6 shows that the relation between the speed and the discharge of a centrifugal pump is somewhat inconsistent. For the case in question, at least, there can be no hard and fast rule governing their relations.

Fig. 7 shows the efficiency of the set, the duty obtained and the coal cost per million gallons for pumping against varying heads. The efficiency, of course, includes the efficiency of the combined pump and engine. This efficiency varies from 45.2 to 56.8 per cent.

The curve shows the cost to be 50 cents per million gallons when the head is zero. This is so because the engine drives a generator

for lighting purposes, the cost of which is thus expressed at 50 cents per million gallons. This amount should be deducted from the coal cost, if the cost for pumping alone is wanted.

Mr. Potts.

EDWIN DURYEA, JR., M. AM. SOC. C. E. (by letter).—The writer wishes to add to the discussion of Mr. Harroun's paper a diagram, Fig. 8, showing a test, made by Mr. L. A. Hicks, of a single-stage centrifugal pump.

Mr. Duryea.

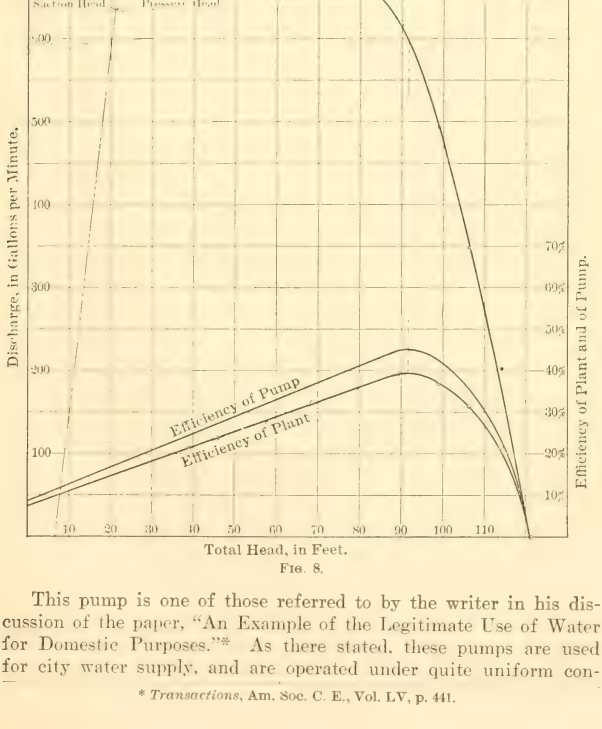

TEST OF NO. 4 CENTRIFUGAL PUMP, BY L. A. HICKS, C.E.
BAKERSFIELD, CAL., SEPT. 6, 1898.
Pressure Head, in Pounds per Square Inch.
Suction Head
Pressure Head
Discharge, in Gallons per Minute.
Efficiency of Plant and of Pump.
Efficiency of Pump
Efficiency of Plant
Total Head, in Feet.

FIG. 8.

This pump is one of those referred to by the writer in his discussion of the paper, "An Example of the Legitimate Use of Water for Domestic Purposes."* As there stated, these pumps are used for city water supply, and are operated under quite uniform con-

* *Transactions*, Am. Soc. C. E., Vol. LV, p. 441.

Mr Duryea. ditions of head or pressure. The pumps are direct-connected to electric motors, thus having a very uniform motive power.

The pressures during the test were created by a partial closing of the discharge valve. The pressure with the valve entirely closed was 49½ lb. per sq. in., and with the valve wide open it was 7 lb. Two check tests (shown by crosses on Fig. 8) were made, in which the pressures were created by a stand-pipe. The suction head was measured by a gauge, and a correction of 1.29 ft. was added for the vertical distance between the centers of suction and pressure gauges. The observed speed during the test varied from 920 to 930 rev. per min. It is said that when the test was made the pump had been in use only two or three weeks.

The pump was a standard commercial one, with no guaranty of efficiency. It was designed by the maker for a pressure of 30 lb. per sq. in., and observations by the writer show that these pumps are being worked against an average or usual pressure of 31 lb. per sq. in., with extremes of, say, 15 and 40 lb. It is worthy of note that the greatest efficiency (45% for the pump and 39.4% for the plant) is at a pressure of 31 lb.

The diagram, Fig. 8, was plotted and the efficiencies worked up from the tabular data of the test recorded by Mr. Hicks. In doing this the efficiency of the electric motor was assumed uniformly at 87%, this being the value given for this motor by the makers.

The diagram is of interest chiefly to those members of the San Francisco Association of Members, Am. Soc. C. E., who heard Mr. Hicks' discussion on Mr. Harroun's paper and the claims then made by the former of usual high efficiencies for commercial centrifugal pumps when designed for operation under uniform conditions.

Mr. Harroun. PHILIP E. HARROUN, M. AM. SOC. C. E. (by letter).—The writer desires to correct a misunderstanding by Mr. Richards. The test of the unit was in no sense an effort to educate the engineering students of the University of California, but was simply a test of a piece of commercial apparatus for the purposes indicated in the paper. The result of this test, together with the results of other tests made by Professor Le Conte, will be published ultimately by the United States Department of Agriculture.

All Mr. Dunham's inquiries are answered specifically in the text of the paper, with the exception of his question relating to the size of the rotor. Detailed prints showing the construction and dimensions have not been available at any time.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1020.

AN ANALYSIS OF GENERAL FLEXURE IN A STRAIGHT BAR OF UNIFORM CROSS-SECTION.*

By L. J. Johnson, M. Am. Soc. C. E.

1. Introduction.—General flexure is to be understood in this paper to include all cases of stress in a right section in which there is normal stress (tension or compression) in any part of the section. Accordingly, it includes (1) pure flexure; (2) flexure combined with tension or compression; and (3) pure tension or pure compression These three are merely particular cases in which the neutral axis is at a zero, an intermediate, or an infinite, distance, respectively, from the center of gravity of the stressed section.

The correlative to general flexure would be general torsion, the latter covering all cases in which tangential stress (shear) is involved, just as the former covers the whole ground of normal stress. Together, they would include all cases of stress. Each may be looked upon as the result of resistance to rotary deformation, the axis of rotation being, in the former case, in the plane of the stressed section, in the latter, normal to that plane.

* Presented at the meeting of April 18th, 1906.

The term analysis is to be understood to mean the study of the distribution of stress over the stressed section.

The attention usually given to this subject is confined to the partial analysis to which almost all writers of English and American textbooks limit themselves. The familiar analysis of flexure, as will be recalled, does not make it clear that the neutral axis will be normal to the plane of the loads only when the plane of the loads is a plane of symmetry of the section, or, more generally speaking, includes a principal axis of inertia of the section. To be sure, this condition does obtain in most cases in practice, but by no means in all, and errors have crept into some of our best books on structural design from a failure to realize the limitations of the familiar analysis. Occasional writers devote a moment's attention to what they call unsymmetrical bending, but, as far as the writer has observed, they always use the complicated method of the principal axes and moments of inertia. Accordingly, this paper is presented, not only on account of the scientific importance of the subject, but also in the hope that it may be of use to practitioners in actual design. The topic treated in this paper has, in recent years, been given much attention by various German writers, particularly Professors Müller-Breslau, Mohr, and R. Land, to whose work the writer renders most appreciative acknowledgment for indispensable aid in his studies. Some years ago the writer published a paper* on the subject, adhering for the most part to the German methods of deduction. But he has since devised methods which, it is believed, will be found much more natural, and they are offered herein with the conviction that some of the most serious complications, and perhaps all the avoidable ones, have been eliminated.

The important equation numbered 7 appeared independently, and as a result of different forms of deduction, in the third edition of Müller-Breslau's *Graphische Statik der Baukonstruktionen,* and in the writer's paper just cited. It appears herein deduced in still a third way. Plate IX and Equations 11, 15, 16, and 18 are believed to have never been published before. What is called herein the *S*-polygon (perhaps the most useful of the results) is a modification of Land's "*W*-Fläche."

Sections 2 to 8 include all that is essential for a working knowl-

Journal of the Association of Engineering Societies, May, 1902.

edge of the subject. Sections 9 to 14 contain tributary matter of a relatively academic nature.

2. Statement of the Problem.—The problem to be solved may be stated in precise terms as follows:

> *Given:* A straight bar of uniform section subject to loads in any plane which includes the longitudinal axis of the body.*
>
> *Required:* I. The intensity of the normal stress at any point of a given right section;
>
> II. The extreme values of the normal stress intensity (called commonly extreme fiber stresses) in the given section;
>
> III. Quick practical means for computing these extreme fiber stresses in all cases;
>
> IV. Similar means, if possible, for indicating which of the familiar rolled-steel sections is to be selected to resist any case of general flexure in order to keep the extreme fiber stresses within prescribed limits.

The satisfaction of the first requirement will clear the way for the others. The latter, though mathematically corollaries to the first, are to the practitioner of paramount importance.

It is to be understood that the section of the body may or may not have an axis of symmetry, and that the forces are not too great to permit the usual assumption of linear distribution of stress. Extreme cases of irregularity of cross-section, which would interfere with the applicability of this assumption, might, perhaps, be imagined, but no section likely to be used in practice is believed to be of this sort.

3. Outline of Method of Procedure.—The external forces on the whole bar being in equilibrium, the force transmitted past the given section of the bar is the resultant, R, of all the forces on one side of the section—an equal, opposite and coincident resultant coming, of course, from the other side and together producing at the same time equilibrium of the body, and stress at the section.

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*This limitation of the location of the loads eliminates torsion—a stress which would not affect the problem, should it exist—and thus helps to fix ideas at the outset.

If the given forces should not be in a plane, as just limited, general flexure would be due only to their components in such a plane, and only such components would enter the problem. The above statement of the data, therefore, is actually general in spite of appearances to the contrary.

Suppose (Fig. 1) the point where R intersects the plane of the section be called K, distant q from the center of gravity, G, of the section. R may be resolved into two components at K, one normal to the section, the other tangential and passing through G. Only the former affects the problem; and it alone, under the designation, N, will henceforth be considered.

Statically speaking, the problem is to resolve N into an infinite number of non-co-planar parallel forces. Pure statics being unequal to the task, the necessary aid is found in the principle of linear distribution of stress, a principle now universally accepted as valid for moderately loaded beams of wood and metal. Stating this prin-

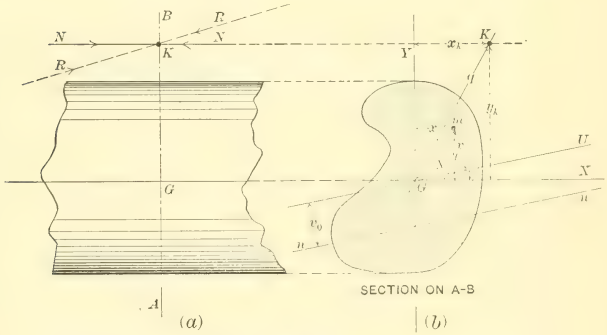

FIG. 1.

ciple and the conditions of equilibrium in algebraic language, there result four simultaneous equations, the solution of which yields the required quantity, stated in terms of the given quantities. This solution once obtained, it will receive further study, with a view to facilitating its application.

4.—Algebraic Statement of the Conditions.—Let the beam have any section, as that of Fig 1(*b*), referred to any convenient pair of rectangular axes, $G\ X$ and $G\ Y$, the origin, G, being the center of gravity of the section. Let f be the required stress-intensity at m, any infinitesimal portion of the section, distant x and y from $G\ Y$

and $G\ X$, respectively. The point, K, the co-ordinates of which will be called x_k, y_k, being, as above stated, the intersection of R with the section, and distant q from G, call λ the inclination of $G\ K$ to $G\ X$. Let $n\ n$ be the locus of points where f is zero (that is, the neutral axis), inclined to $G\ X$ by the angle, α. $G\ U$ is a line drawn through G parallel to $n\ n$. The distance between $n\ n$ and $G\ U$ will be called v_o.

The algebraic statement of the principle of linear distribution may conveniently be written

$$f = f_o - \frac{f_o}{v_o}\ v,$$

in which f_o is the constant value of f at all points of $G\ U$, and v is the distance of m from $G\ U$. The lengths, v and v_o, are + or — according as they are measured from $G\ U$ toward or away from K. Accordingly, f_o and $\frac{f_o}{v_o}$ are placed in the equation with opposite signs, and + is selected for the former. Then, as will be seen from Equation 8, a positive result for f will indicate that the stress at m is of the same sign as if N were applied at G, and *vice versa*. Referring m to $G\ X$ and $G\ Y$, by substituting $y \cos.\ \alpha - x \sin.\ \alpha$ for v, the equation becomes

$$f = f_o - \frac{f_o}{v_o}\ (y \cos.\ \alpha - x \sin.\ \alpha) \ldots\ldots\ldots\ldots\ldots (1)$$

Here, f_o, v_o, and α are unknowns, and three more equations are required for their evaluation. The conditions of equilibrium are now brought into use.

Expressing the infinitesimal area, m, as $d\ x\ d\ y$, an unbalanced single force will be precluded under the conditions if

$$\int\int f\ d\ x\ d\ y = N \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

and, similarly, an unbalanced couple will be precluded if

$$\int\int x\ f\ d\ x\ d\ y = N\ x_k \ldots\ldots\ldots\ldots\ldots (3)$$

and

$$\int\int y\ f\ d\ x\ d\ y = N\ y_k \ldots\ldots\ldots\ldots\ldots (4)$$

With these three equations the unknowns can be eliminated from Equation 1, and the expression for f will be left in proper shape for discussion and use.

5. Deduction of the Desired Equations.—Substituting* the value of f from Equation 1 in each of the Equations 2, 3 and 4, and noting that

$$\int\int dx\,dy = A, \text{ the area of the section,}$$

$$\int\int x\,dx\,dy = 0, \text{ since } GY \text{ is a gravity axis,}$$

$$\int\int y\,dx\,dy = 0, \text{ since } GX \text{ is a gravity axis,}$$

$$\int\int y^2\,dx\,dy = I_x\dagger, \text{ the moment of inertia of the section referred to } GX,$$

$$\int\int x^2\,dx\,dy = I_y\dagger, \text{ the moment of inertia of the section referred to } GY,$$

$$\int\int xy\,dx\,dy = J\dagger, \text{ the product of inertia of the section referred to } GX \text{ and } GY.$$

---

* This substitution being a highly important step, the algebraic reduction is stated as follows:

Equation 2, with $f_o - \frac{f_o}{v_o}(y\cos.\,\alpha - x\sin.\,\alpha)$ substituted for f, becomes

$$\int\int \left\{f_o - \frac{f_o}{v_o}(y\cos.\,\alpha - x\sin.\,\alpha)\right\} dx\,dy = N,$$

that is,

$$f_o\int\int dx\,dy - \frac{f_o}{v_o}\cos.\,\alpha\int\int y\,dx\,dy + \frac{f_o}{v_o}\sin.\,\alpha\int\int x\,dx\,dy = N.$$

Inserting values of the integrals stated above, this becomes

$$A f_o = N,$$

whence Equation 5.

Equation 3, upon substitution of $f_o - \frac{f_o}{v_o}(y\cos.\,\alpha - x\sin.\,\alpha)$ for f, becomes

$$\int\int \left\{f_o - \frac{f_o}{v_o}(y\cos.\,\alpha - x\sin.\,\alpha)\right\} x\,dx\,dy = N x_k,$$

that is,

$$f_o\int\int x\,dx\,dy - \frac{f_o}{v_o}\cos.\,\alpha\int\int xy\,dx\,dy + \frac{f_o}{v_o}\sin.\,\alpha\int\int x^2\,dx\,dy = N x_k.$$

Inserting the stated values of the integrals as before, this becomes

$$-\frac{f_o J\cos.\,\alpha}{v_o} + \frac{f_o I_y}{v_o}\sin.\,\alpha = N x_k.$$

or

$$\frac{f_o}{v_o}(J\cos.\,\alpha - I_y\sin.\,\alpha) = N x_k.$$

whence

$$\frac{f_o}{v_o} = \frac{N x_k}{J\cos.\,\alpha - I_y\sin.\,\alpha}, \text{ which is part of Equation 6.}$$

Similarly, by substituting Equation 1 in Equation 4 it falls out that

$$\frac{f_o}{v_o} = \frac{N y_k}{I_x\cos.\,\alpha - J\sin.\,\alpha},$$

which completes Equation 6.

† Computation of I_x, I_y, and J.—The computation of I_x and I_y involves, in practice, only the application of the familiar, $I' = I_o + A h^2$. For all the structural shapes, the

it will appear that

$$f_o = \frac{N}{A} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

(showing that the unit stress at the center of gravity is always the same as it would be if N were applied there), and that

$$-\frac{f_o}{v_o} = \frac{N x_k}{J \cos.\ \alpha - I_y \sin.\ \alpha} = \frac{N y_k}{I_x \cos.\ \alpha - J \sin.\ \alpha} \qquad \ldots\ldots\ldots (6)$$

Solving Equation 6 yields, noting that $\frac{y_k}{x_k} = \tan.\ \lambda$,

$$\left.\begin{aligned} \tan.\ \alpha &= \frac{I_x - J \tan.\ \lambda}{J - I_y \tan.\ \lambda} \\ &= \frac{I_x \cot.\ \lambda - J}{J \cot.\ \lambda - I_y} \\ &= \frac{I_x \cos.\ \lambda - J \sin.\ \lambda}{J \cos.\ \lambda - I_y \sin.\ \lambda} \end{aligned}\right\} \qquad \ldots\ldots\ldots\ldots (7)$$

the various forms of the second member being convenient in special cases. The first two are usually the simplest to use, but either may become indeterminate on the substitution of special values of λ. If one of these fails in this way, the other or the third can be used.

Substituting Equations 5 and 6 in Equation 1, there results

$$f = \frac{N}{A} + \frac{N x_k (y - x \tan.\ \alpha)}{J - I_y \tan.\ \alpha} \qquad \ldots\ldots\ldots\ldots (8)$$

or

$$f = \frac{N}{A} + \frac{N y_k (y - x \tan.\ \alpha)}{I_x - J \tan.\ \alpha} \qquad \ldots\ldots\ldots\ldots (9)$$

Equations 8 and 9, if written in terms of q, will express f as the result of combining two quantities which represent, respectively, the contributions to f from a force, N, at G, and the couple, Nq, that is, from a force and couple into which any N at K can be resolved.

I_x and I_y are given outright by the handbooks. J is not so familiar a quantity and is not mentioned by the handbooks, yet its computation in practical cases is very easy. From the nature of its definition,

$$J = \int \int x y \, dx \, dy,$$

it is evident that, for a pair of axes, one of which is an axis of symmetry of the section, $J = 0$. It is also zero for the principal axes of inertia. In all ordinary cases it can be computed by the aid of the simple transformation formula, exactly analogous to $I = I_o + A h^2$,

$$J' = J_o + A k h,$$

in which J' is the J of the section referred to any rectangular co-ordinates, $O X$ and $O Y$, k and h are co-ordinates of the center of gravity, G, of the section; J_o is the J referred to axes parallel to $O X$ and $O Y$, with origin at G. Numerical illustrations of the computation of J will be found in Sections 5 and 6. J will be positive or negative according as the preponderance of the section is in the first and third, or second and fourth quadrants.

I_x, I_y, and J are all quantities of the fourth degree, and are expressed in biquadratic inches or biquadratic centimeters, etc.

Noting that x_k and y_k are $q \cos. \lambda$ and $q \sin. \lambda$, respectively, Equations 8 and 9 become, accordingly,

$$f = \frac{N}{A} + \frac{N q (y - x \tan. \alpha) \cos. \lambda}{J - I_y \tan. \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8')$$

and

$$f = \frac{N}{A} + \frac{N q (y - x \tan. \alpha) \sin. \lambda}{I_x - J \tan. \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9')$$

Indeterminations in all these equations can be evaluated by the aid of trigonometric transformations such as were used in stating the second member of Equation 7, or by selecting between Equations 8 and 9, or between Equations 8′ and 9′.

The value of f is now stated in the way most convenient for the common practical problem, which is to find the maximum value of f, that is, the value of f for the m most remote from $n\ n$. This particular m will be the m most remote from $G\ U$ on the same side of $G\ U$ as K, as will be shown in Section 9. The routine in the solution of this problem will consist of:

(*a*) The solution of Equation 7;

(*b*) The identification of the most remote m, by sketching or drawing $G\ U$—a line through G the slope of which is tan. α;

(*c*) The substitution of the tan. α and x, y for this m in Equations 8 or 9.

Parts I and II of the requirements of the problem are now accomplished. A numerical example will be worked out, by way of illustration, and then attention will be directed to a study of the equations just obtained, with a view to meeting requirements III and IV of the problem (Section 2).

It will be worth while, in passing, to combine Equation 7 with Equations 8 or 9, and with Equations 8′ or 9′, and record expressions for f in terms of known quantities, exclusively. They are found to be:

$$f = \frac{N}{A} + \frac{N (y_k I_y - x_k J)\, y + N (x_k I_x - y_k J)\, x}{I_x I_y - J^2} \quad \ldots\ldots\ldots (10)$$

or

$$f = \frac{N}{A} + N q \frac{(I_y \sin. \lambda - J \cos. \lambda)\, y + (I_x \cos. \lambda - J \sin. \lambda)\, x}{I_x I_y - J^2} \quad \ldots\ldots (10')$$

Numerical Example.—Problem.—A single 4 by 3 by $\frac{8}{8}$-in. angle, as a member of a framework, is attached at each end to a $\frac{1}{2}$-in. gusset

by a line of rivets of 2¼-in. gauge, through the 4-in. leg only. Suppose the direct stress is a thrust, P, acting in the line of the rivets and at mid-thickness* of the gusset. What is the maximum compressive unit stress in the angle?

Solution.—The solution consists of mere substitution in Equations 7 and 8 or 9, the determination of J offering the only unusual difficulty.

Showing the data in a sketch (Fig. 2), in which the section is referred to the most convenient axes, the section is divided into two

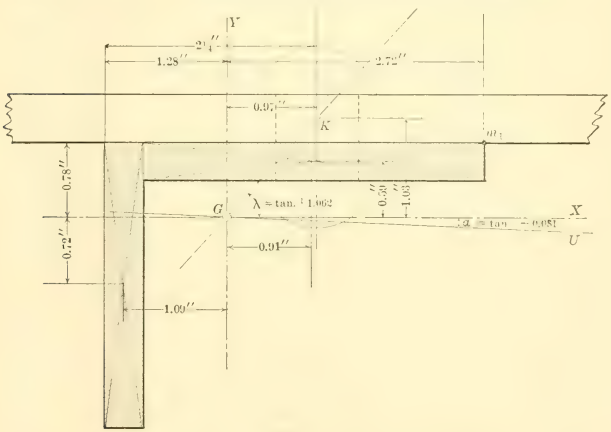

Fig. 2.

rectangles, whose centers of gravity are at (— 1.09, —0.72) and (0.91, 0.59), and whose areas are $3 \times \frac{3}{8} = 1.125$, and $3\frac{5}{8} \times \frac{3}{8} = 1.359$, respectively. Then, by $J' = J_o + A\ k\ h$, of the foot-note, p. 174, noting that J_o of both rectangles is zero,

$$J = 1.125 \times -1.09 \times -0.72 + 1.359 \times 0.91 \times 0.59 = 1.61 \text{ in.}^4$$

The Carnegie book gives $I_x = 1.92$ in.^{4}, $I_y = 3.96$ in.^{4}, $A = 2.48$ in.

Then, by Equation 7, noting that $\tan.\ \lambda = \frac{1.03}{0.97} = 1.062$,

$$\tan.\ \alpha = \frac{1.92 - 1.61 \times 1.062}{1.61 - 3.96 \times 1.062} = \frac{0.21}{-2.60} = -0.081.$$

* This problem is solved, upon other assumptions as to the point of application of the thrust, at the close of Section 9.

$G\ U$, accordingly, is in the second and fourth quadrants, and the extreme fibers of the angle will be the most remote corners in the first and third quadrants, with m_1 as the fiber subject to maximum compressive stress. The co-ordinates of m_1 are (2.72, 0.78), and, observing that $y_k = 1.03$, substitution in Equation 9 yields

$$f = \frac{N}{2.48} + N\frac{1.03\left(0.78 - \overline{2.72 \times -0.081}\right)}{1.92 - \overline{1.61 \times -0.081}}$$
$$= 0.40\,N + 0.50\,N = 0.90\,N, \text{ the required answer.}$$

That is, the effect of the eccentricity is to make the extreme stress more than double the average. If the direct stress, N, is 10 000 lb., $f_{max.}$ will be 9 000 lb. per sq. in. Equation 8, of course, would give the same result.

If the strut is so long as to deflect materially, the section at the point of maximum sidewise deflection will have a longer q and a different λ from that in the preceding solution. For such a case the method would be precisely as above, after the change in direction and length of $G\ K$ had been estimated—a process outside the scope of this paper.

6. Special Forms of Expressions for "f."—In pure normal stress, when q and x_k and y_k, all vanish, Equations 8, 9, and 10 reduce to

$$f = \frac{N}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10a)$$

as they should.

In pure flexure, $N = 0$, $q = \infty$, and Nq is the value of the bending couple commonly written M. In this case Equations 8, 9 and 10, take the forms

$$f = \frac{M\,(y - x\tan.\,\alpha)\cos.\,\lambda}{J - I_y \tan.\,\alpha.} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8a)$$

$$f = \frac{M\,(y - x\tan.\,\alpha)\sin.\,\lambda}{I_x - J\tan.\,\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9a)$$

$$f = M\frac{(I_y \sin.\,\lambda - J\cos.\,\lambda)\,y + (I_x\cos.\,\lambda - J\sin.\,\lambda)\,x}{I_x I_y - J^2} \quad \ldots\ldots (10b)$$

Then, if $\lambda = 90°$, and $\alpha = 0°$, as it will if J is zero, these equations all reduce to the familiar but highly special

$$f = \frac{M\,y}{I_x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10c)$$

The very common applicability of this equation arises from the fact that the rectangular beams and beams with I, C, or T-sections are naturally referred to rectangular axes for which J is zero, and are loaded in a plane containing one of these axes.

Numerical Examples.—1.—A 5 by $3\frac{1}{4}$ by $\frac{1}{2}$-in. Z-bar acts as a purlin on a roof having a slope of 30°, its top flange projecting toward the ridge. It supports vertical loads which cause a maximum flexure of M inch-pounds. Required, the extreme fiber stresses.

Solution.—Showing data, and taking axes as in Fig. 3, I_x and

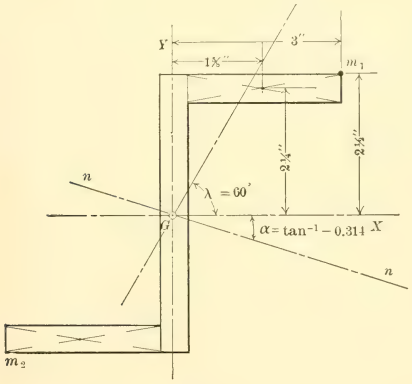

Fig. 3.

I_y are given by steel handbooks as 19.19 in.⁴ and 9.05 in.⁴, and J, computed in a manner similar to that of the angle iron of the preceding section, is found to be $2(2.75 \times 0.5 \times 2.25 \times 1.625) = +\ 10.05$ in.⁴

By Equation 7, tan. λ being tan. 60° = 1.732,

$$\text{tan. } \alpha = \frac{19.19 - 10.05 \times 1.732}{10.05 - 9.05 \times 1.732} = -0.317$$

The extreme fibers are located accordingly at m_1 and m_2 the co-ordinates of which are ± 3.0 and ± 2.5. Using the values for m_1, and substituting in Equation 9a.

$$f = \frac{(2.5 - 3.0 \times -0.317) \times 0.866}{19.19 - 10.05 \times -0.317} M = 0.134\ M \text{ lb. per sq. in.}$$

The f for m_2 would differ from this only in sign, and $\pm 0.134\ M$ is the required answer.

2.—Suppose the Z-bar of the preceding case replaced by a 6 by 12-in. rectangle with the 12-in. side normal to the roof slope. Find the fiber stresses.

Solution.—Taking $G\ X$ parallel to the short side, J is zero, and I_x and I_y are 864 and 216 in.^{4}, respectively. Hence, by Equation 7,

$$\text{tan. } \alpha = \frac{864}{-216 \times 1.732} = -2.31$$

The extreme fibers are ± 3.0 and ± 6.0. Inserting in Equation 9a

$$f = \pm \frac{(6 - 3 \times -2.31) \times 0.866}{864} M = \pm 0.013\ M \text{ lb. per sq. in.,}$$

the required answer.

7. The S-polygon of a Section.—As has been observed, any case of flexure may be treated as a combination of direct normal stress and pure flexure, either of which may, of course, be zero. The treatment of the pure flexure is the only part offering much difficulty. The difficulty with pure flexure is specially great when the extreme value of f is to be determined, and the proper values of x and y, substituted in Equation 10b, have to be computed in advance. The diminution of the difficulty in this most important practical problem is the next field of inquiry.

Inspection of Equations 8a, 9a and 10b reveals that the f for any m may be obtained by dividing M by a quantity in which, for a given λ, the only variables are the co-ordinates of m. This quantity, in its most general form, is, from Equation 10b,

$$S = \frac{I_x I_y - J^2}{(y I_y - x J) \sin. \lambda + (x I_x - y J) \cos. \lambda} \quad \ldots\ldots\ldots (11)$$

a quantity which might be called the flexure modulus of the section for the point, x, y, or, more briefly, the section modulus for x, y. Special values of it for extreme positions of m and for special values of λ are what are given in the steel manufacturers' handbooks, in the case of I-beams, channels and tees, as the section moduli of the shapes concerned. In this paper it will be referred to simply as S, occasionally with a special suffix.

S is a quotient obtained by dividing a quantity of the eighth degree by one of the fifth. It is, therefore, of the third degree. It may

be looked upon as the statical moment of an area.* Thus, only can M divided by S yield a stress-intensity. In the special case in which $\lambda = 90°$, and J is zero, S takes the familiar form

$$S_a = \frac{I_x}{y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

Returning to the examination of Equation 11, it at once appears that, for a fixed m and varying λ, that equation is the polar equation of a straight line, that is, a line the radius vector of which, at any inclination, λ, would be proportional to the S for that λ. That is, the S-line, if it may be so called, is the simplest possible graphical exhibit for the values of S for any one m for all values of λ. This at once suggests the determination of the S-lines for all points of the section which might be extreme fibers of the section. As will be seen in what follows, these S-lines will define a surface which may be called the S-polygon of the section. With the S-polygon given, merely drawing a line through G with the inclination, λ, defines two radii vectores, S' and S'', measured from G toward and away from K, respectively. S' and S'' being measured to the proper scale, the most important values of f in the most difficult case can be had by mere substitution in

$$f' = \frac{N}{A} + \frac{Nq}{S'} = \frac{N}{A} + \frac{M}{S''} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

$$f'' = \frac{N}{A} + \frac{Nq}{S''} = \frac{N}{A} - \frac{M}{S''} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

which are perfectly general expressions for the extreme values of f. Of course, in pure flexure, N vanishes, and they become

$$f' = \frac{M}{S'} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13')$$

$$f'' = \frac{M}{S''} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14')$$

8.—Construction of the S-polygon.—Referring Equation 13 to rectangular co-ordinates by substituting x and y for $S \cos. \lambda$, and $S \sin. \lambda$, respectively, we have

$$y = \frac{x_m I_x - y_m J}{x_m J - y_m I_y} x - \frac{I_x I_y - J^2}{x_m J - y_m I_y} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

a line which is parallel to the neutral axis (Equation 17 in Section 9) for a K at x, y, on the same side of G with K (that is, on the opposite side of G from $n\ n$), and, if the same units be selected to rep-

* The units in which it is expressed may be called inch square-inches, following the analogy with inch-pounds. Inch square-inches will be abbreviated to inches^2 or in.^{2}

resent third-degree units as to represent simple length, A times as far from G as $n\ n$ is.

Of course, the values of x and y, for all points on the perimeter of the given section could be substituted for (x_m and y_m) in Equation 15, the corresponding lines plotted, and the space within all these lines taken as the S-polygon. This might have to be resorted to for parts of the section bounded by curved lines, but it can be done more easily in the case of the familiar rolled steel shapes. In these cases the extreme m's will all lie upon a small number of straight lines which bound, but do not, even if produced, cross the section. These lines will constitute what may be called the circumscribing polygon. The S-polygon will have a side corresponding to each apex of the circumscribing polygon. These sides will be S-lines, intersecting and forming apices of the S-polygon whenever λ has such a value as will cause two adjacent apices of the circumscribing polygon to be extreme fibers at the same time. This will happen, of course, whenever the neutral axis is parallel to the side of the circumscribing polygon.

Locating and connecting the apices will usually be the most convenient way of constructing the S-polygon. The co-ordinates of an apex of the S-polygon can be determined by substituting the co-ordinates (x_a, y_a) and (x_b, y_b) of two successive apices, A and B, of the circumscribing polygon in Equation 15, and solving the two resulting equations for the co-ordinates of the intersection of the resulting S-lines. Performing this operation will yield for the co-ordinates (x_{ab}, y_{ab}) of the apex of the S-polgyon corresponding to the side, $A\ B$,

$$\left.\begin{aligned} x_{ab} &= \frac{(x_a - x_b)\,J - (y_a - y_b)\,I_y}{x_a\,y_b - x_b\,y_a} \\ y_{ab} &= \frac{(x_a - x_b)\,I_x - (y_a - y_b)\,J}{x_a\,y_b - x_b\,y_a} \end{aligned}\right\} \quad \ldots\ldots\ldots (16)$$

If, as very commonly happens, the side of the circumscribing polygon is parallel to an axis of reference, these expressions become materially simplified. If, for instance, it is parallel to the X-axis, $y_a = y_b$, and Equation 16 becomes

$$\left.\begin{aligned} x_{ab} &= \frac{J}{y_a} \\ y_{ab} &= \frac{I_x}{y_a} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16')$$

If the line is parallel to the Y-axis, $x_a = x_b$, and Equation 16 becomes

$$\left.\begin{aligned} x_{ab} &= \frac{I_y}{x_a} \\ y_{ab} &= \frac{J}{x_a} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16'')$$

The actual S-polygon, as a fixed geometrical property of a section, and one which can be constructed with great ease any time (see numerical examples below) for the simpler sections, and can be constructed once for all and kept on file for the more difficult sections, is presented as a satisfaction of the requirements of III and IV of Section 2.

The actual methods of computation and use of the S-polygon will be illustrated in the following numerical examples.

Numerical Examples.—Required the S-polygon for each of the four following sections, viz.:

(*a*) 6 by 12-in. rectangle;

(*b*) 8-in. 18-lb. I-beam;

(*c*) 10-in. 20-lb. channel;

(*d*) 5 by ½-in. Z-bar.

Solution.—The natural axes of reference for all these shapes are the gravity axes parallel to the principal dimensions, and shown in Figs. 4, 5, 6, and 7. In each of the first three sections at least one of the axes is an axis of symmetry and J will be zero, and, consequently, the simple Equations 16′ and 16″ will suffice. For the Z-bar, however, Equation 16, in its general form, will be needed. The sides and apices of the S-polygons will be designated with the lower-case letters of the corresponding apices and sides of the circumscribing polygon.

The four shapes will now be taken up in order.

(*a*)—The given 6 by 12-in. rectangle is shown as $A\ B\ C\ D$ in Fig. 4.

$$I_x = \frac{b\,d^3}{12} = \frac{6 \times 12 \times 12 \times 12}{12} = 864 \text{ in.}^4;$$

$$I_y = \frac{b^3\,d}{12} = \frac{6 \times 6 \times 6 \times 12}{12} = 216 \text{ in.}^4;$$

$$J = 0.$$

It will suffice to compute the co-ordinates of the apices, $a\ b$ and $b\ c$; symmetry will locate the other two, $c\ d$ and $a\ d$.

The co-ordinates of A, B, and C are $(-3, 6)$, $(3, 6)$ and $(3, -6)$. These are the (x_a, y_a), (x_b, y_b), etc., of Equations 16, 16′, and 16″.

By Equation 16′, $x_{ab} = \frac{J}{y_a} = \frac{0}{6} = 0$; $y_{ab} = \frac{I_x}{y_a} = \frac{864}{6} = 144$ in.^{3}

By Equation 16″, $x_{bc} = \frac{I_y}{x_b} = \frac{216}{3} = 72$ in.^{3}; $y_{bc} = \frac{J}{x_b} = \frac{0}{3} = 0$.

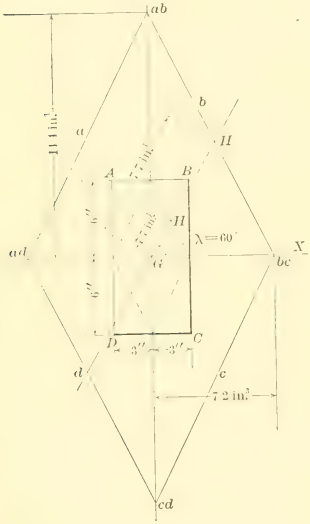

Fig. 4.

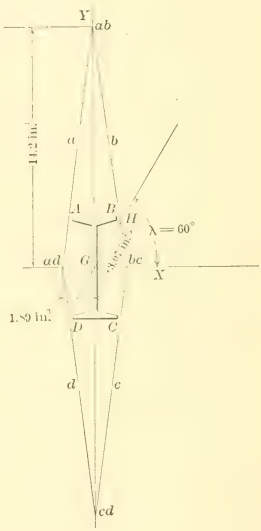

Fig. 5.

The four apices are, accordingly, $(0, \pm 144)$ and $(\pm 72, 0)$, and are plotted in Fig. 4 to an arbitrarily selected scale.

Remarks.—The sides of this S-polygon are parallel to the diagonals of the rectangle, and an S-polygon, therefore, could be constructed by merely connecting the middle points of adjacent sides of the rectangle. Such a polygon is shown dotted in Fig. 4. The scale of this S-polygon is readily determined by computing the numerical value, $\frac{b\,d^2}{6}$, of its vertical semi-diagonal.

The apices are seen to be on the axes at distances from G which represent the familiar section moduli in the planes of the axes along which they are laid off. Consequently, in dealing with rectangles in general, or similar simple sections, no formal use of Equations 16′ and 16″ is necessary. The section moduli once known, they are laid off to convenient scale along the proper axes, and the apices thus determined are connected. This would apply to any section referred to rectangular gravity axes for which J is zero. This method will be exemplified in the next two cases.

(*b*)—For the 8 in., 18-lb. I-beam, the steel handbooks give the section modulus for the axis, $G\ X$, as 14.2 in.^{3} The other is found by dividing the I_y by half the flange width—both given by the handbooks, that is, $\frac{3.78}{2.0} = 1.89$ in.^{3} Laying off the 14.2 upward and downward from G, and the 1.89 to the right and left of G, the S-polygon of Fig. 5 is established.

(*c*)—For the 10-in., 20-lb. channel, the two section moduli, 15.7 and 1.34 in.^{3}, given by the handbooks, must be supplemented by a third one, since $G\ Y$ is not an axis of symmetry. This third one is I_y divided by the distance from G to the back of the channel. Using the handbook data, this is found to be $\frac{2.85}{0.61} = 4.67$ in.^{3} Laying off 15.7 upward and downward from G, 1.34 from G along $G\ X$ and in the same direction as the projecting flanges, and 4.67 along $G\ X$ in the opposite direction, the S-polygon of Fig. 6 is located.

(*d*)—For the 5 by $\frac{1}{2}$-in. Z-bar, the handbooks furnish $I_x = 19.19$ in.^{4}, $I_y = 9.05$ in.^{4}, and, in Section 7, J was found to be 10.05 in.^{4} J will be positive for the section as drawn in Fig. 7. The three apices, $a\ b$, $b\ c$, and $c\ d$, will be located by computation, and the other three, $d\ e$, $e\ f$, $f\ a$, will follow by symmetry. The coordinates of A, B, C, and D are (— 0.25, 2.50), (3.0, 2.50), (3.0, 2.0), and 0.25, — 2.50).

By Equation 16′,

$$x_{ab} = \frac{10.05}{2.50} = 4.02 \text{ in.}^3; \quad y_{ab} = \frac{19.19}{2.50} = 7.68 \text{ in.}^3.$$

By Equation 16″,

$$x_{bc} = \frac{9.05}{3.0} = 3.02 \text{ in.}^3; \quad y_{bc} = \frac{10.05}{3.0} = 3.35 \text{ in.}^3.$$

By Equation 16,

$$x_{cd} = \frac{(3.0 - 0.25)10.05 - (2.0 - 2.5)9.05}{3.0 \times -2.5 - 0.25 \times 2.0} = 1.63 \text{ in.}^3;$$

$$y_{cd} = \frac{(3.0 - 0.25)19.19 - (2.0 - 2.5)10.05}{3.0 \times -2.5 - 0.25 \times 2.0} = -0.94 \text{ in.}^3.$$

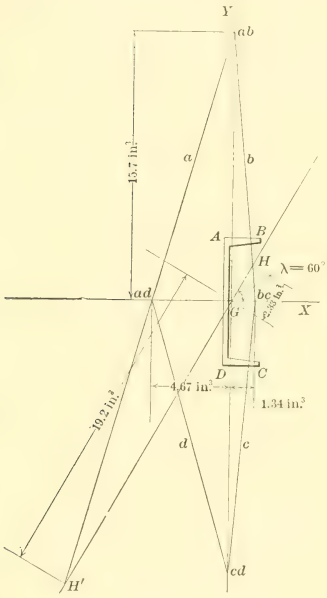

FIG. 6.

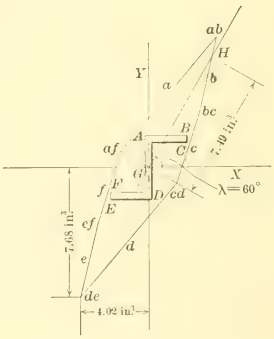

FIG. 7.

These and the same with signs reversed establish the desired S-polygon plotted in Fig. 7. Similarly, any section can be treated.

9. Examples of the Use of the S-Polygon.—The following numerical examples will illustrate some of the uses of the S-polygon.

Problem 1.—Find the extreme fiber-stress in each of the four sections of Figs. 4 to 7, assuming each to be acting as a purlin upon a roof inclined 30° to the horizon and set with its G X-axis parallel to the roof slope. Assume a flexure of M, in pounds, in a vertical

plane, and assume that each purlin is set so as to make B the highest corner.

Solution.—Draw a line with a slope, $\lambda = (90° - 30°) = 60°$ (representing the trace of the plane of loads) through G in each S-polygon of Figs. 4 to 7, and scale off the distance, $G\,H$, along this line to the S-polygon perimeter, taking the shorter of the two lengths when there is a difference. The results for the four sections are as dimensioned 77, 3.07, 2.33, and 7.49 in.^{3}, respectively. The results, accordingly, are by Equation 13, N being zero:

For the rectangle:

$$f_{max.} = \pm \frac{M}{77} = \pm 0.013\ M \text{ lb. per sq. in.}$$

For the 8-in., 18-lb. I-beam:

$$f_{max.} = \pm \frac{M}{3.07} = \pm 0.325\ M \text{ " " " "}$$

For the 10.-in., 20-lb. channel:

$$f_{max.} = + \frac{M}{2.33} = + 0.428\ M \text{ " " " "}$$

For the 5 by $\frac{1}{2}$-in. Z-bar:

$$f_{max.} = \pm \frac{M}{7.49} = \pm 0.134\ M \text{ " " " "}$$

Here are verified the results of the numerical example of Section 6. Incidentally, note the great economy (for such a load) of the Z-bar over the heavier channel and I-beam.

Problem 2.—In each of the sections in the preceding problem, which are the fibers subject to the stresses given?

Solution.—In the rectangle and I-beam, H might be taken on either the b-line or the d-line. Hence, B and D are both extreme fibers in these cases. In the channel, similarly, B is indicated, and in the Z-bar, B and E.

Problem 3.—In the channel of Problem 1, what is the magnitude and nature of the stress at the point, A, under the given load?

Solution.—Extending $G\,H$ to the intersection, H', with the a-line produced, $G\,H'$ is scaled off as 19.2 in.^{3} Hence, $f_A = \frac{M}{19.2} = 0.052\ M$. To determine whether it is tension or compression, observe that N, though zero in amount, may be treated as a compressive force at infinity on $G\,H$ above G, or as a tensile force at infinity below G.

Assume the former, then H' is on the opposite side of G from K, and f_A, accordingly (Section 7), is of opposite sign to N, and, therefore, is tensile. The same result follows, of course, from the opposite assumption.

Problem 4.—In the Z-bar of Fig. 7, what is the most favorable plane of loading, and what will then be the position of the neutral axis?

Solution.—The most favorable plane of loading is obviously that yielding the maximum S. This, by inspection of the figure, would be a plane the trace of which would pass through $a\ b$ and $d\ e$, and have for a slope $\tan.^{-1} \frac{7.68}{4.02}$. This trace, passing through $a\ b$ and $d\ e$, indicates that A and B, as well as D and E, are then all extreme fibers. Hence, the neutral axis is then parallel to the flanges, $A\ B$ and $D\ E$.

Similarly, the least favorable plane of loads might be located; obviously, it will be perpendicular to the a-line and d-line.

Problem 5.—Through what extremes would $f_{max.}$ pass, if the Z-bar of Problem 1 were to make a complete revolution about its longitudinal axis, the load remaining vertical?

Solution.—Scaling off the extreme values of S indicated in the preceding problem, they are found to be 8.68 and 1.85 in.^{3} The extremes of $f_{max.}$ are $\frac{M}{8.68} = 0.116\ M$, and $\frac{M}{1.85} = 0.540\ M$.

Problem 6.—What is the least favorable plane of loads for the rectangle of Fig. 4?

Solution.—Minimum S will occur for planes normal to either diagonal of the rectangle.

Problem 7.—What plane of loads will make the neutral axis vertical in the Z-bar of Fig. 7?

Solution.—If the neutral axis is vertical, $B\ C$ and $E\ F$ will be the extreme fibers, and the plane of loads must pass through $b\ c$ and $e\ f$.

Problem 8.—A channel is to be selected to act as a purlin; trusses 10 ft. on centers; slope of roof 30° to horizontal; purlins 6 ft. apart in plan; loads, dead 10 lb. and snow 20 lb. per sq. ft. of plan, and wind 25 lb. per sq. ft. of roof surface; fiber stress not to

exceed 16 000 lb. per sq. in. for the worst combination, considering (*a*) that the maximum snow and wind may act simultaneously; (*b*) that they may not so act.

Solution.—The loads per linear foot upon the purlin for the three

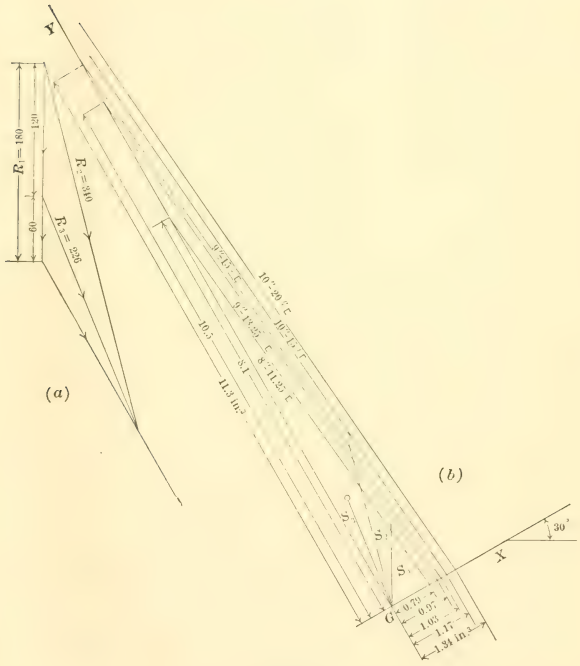

FIG. 8.

loads are, noting that the distance between purlin centers on the slope is 6.93 ft.,

$w_d = 6 \quad \times 10 = \quad 60$ lb. per lin. ft., vertical,

$w_s = 6 \quad \times 20 = 120$ " " " " "

$w_w = 6.93 \times 25 = 173.2$ " " " " normal to roof slope.

Fig. 8*a* is a force-magnitude polygon showing the important result-

ants, R_1, R_2 and R_3 in magnitude and direction. They are the resultants of w_a and w_s, of w_a, w_s, w_w, and of w_a and w_w, respectively. They give rise to three different flexures, M_1, M_2 and M_3, in their respective planes, and call for three different values of S: S_1, S_2 and S_3, in those planes.

$$S_1 = \frac{M_1}{16\,000} = \frac{180 \times 10 \times 10 \times 12}{8 \times 16\,000} = 180 \times 0.0094 = 1.69 \text{ in}^3.$$

Similarly, S_2 and S_3 are 3.19 and 2.12 in.^{3}, respectively.

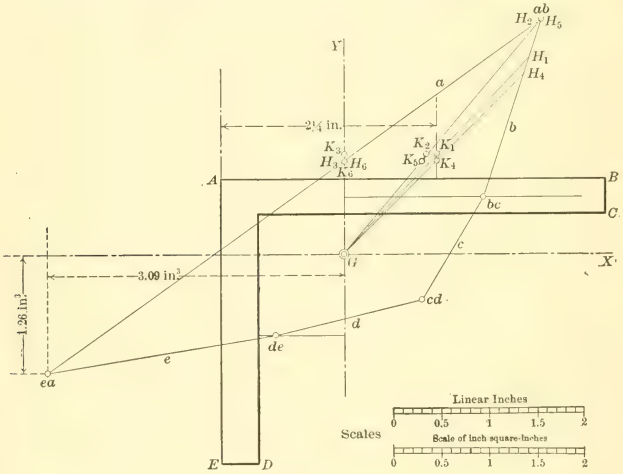

FIG. 9.

On inspection of Fig. 6 it is seen that a single side (that pertaining to B) of the S-polygon for each channel will suffice. Accordingly, this side is plotted for a promising list of channels, using only such data as the handbook furnishes. S_1, S_2 and S_3 are laid off from G parallel to the resultants to which they respectively correspond. This work is shown in Fig. 8*b*. Condition (*a*) calls for a channel which can furnish S_1 and S_2, and condition (*b*) requires S_1 and S_3. A glance at the sketch shows that for (*a*) a 10-in. 15-lb. channel is required, and for (*b*) a 9-in. 13.25-lb. channel. The latter is so close a fit that one of the next two larger sizes might well be preferred, in view of the sensitiveness of the section to a slight change in position.

Problem 9.—Suppose the 4 by 3 by $\frac{3}{8}$-in. angle of the example at the close of Section 5 to be exposed to a thrust, P (= 18 000 lb.), applied successively at the six different K's, shown in Fig. 9. The first three are at mid-thickness of a $\frac{1}{2}$-in. gusset, and the second three similarly on the center plane of a $\frac{3}{8}$-in. gusset. In each set, the K's are, respectively, at gauge $2\frac{1}{4}$ in., at the most favorable gauge, and opposite G. Required the $f_{max.}$ for each case.

Solution.—Here are really six problems, each equal in complexity to that at the close of Section 5. In fact, the first one is that very problem, here to be solved again for comparison. The S-polygon simplifies matters so greatly that the six problems will be taken up together. It is merely a question of getting the M's and S''s for substitution in Equation 13. S' is, in each case, $G\ H$, as usual, and $M = P\ q$, where q is $K\ G$. The six q's and S''s are scaled from Fig. 9, where the section and its S-polygon are plotted, and the substitution is carried out in Table 1.

TABLE 1.

Case.	q.	S'.	$\frac{M}{S''} = \frac{P\ q}{S''}$	$\frac{P}{A}$	$f_{max.} = \frac{P}{A} + \frac{M}{S''}$	Location of $f_{max.}$
K_1	1.41 in.	2.82 in.^{3}	9 000	7 200	16 200	B
K_2	1.35 "	3.21 "	7 600	7 200	14 800	$A\ B$
K_3	1.03 "	0.97 "	19 100	7 200	26 300	A
K_4	1.37 "	2.66 "	9 300	7 200	16 500	B
K_5	1.27 "	3.21 "	7 100	7 200	14 300	$A\ B$
K_6	0.97 "	0.97 "	18 000	7 200	25 230	A

The location of the fiber, subject to $f_{max.}$ in each case, is indicated by the letter of the S-line on which H falls, and is recorded in the table as a matter of interest.

The fluctuation of f is seen to be considerable with these changes in the position of K. In an actual frame these variations would doubtless be less serious, but would be diminished only at the expense of secondary stresses in the neighboring members. These stresses would be of a flexural or torsional character very difficult to estimate.

Section 10.—Concerning S-Polygons of Specially Important Sections.—Table 2 gives the co-ordinates of the apices of the S-polygons for the standard American **Z**-bars, and Plate IX shows these same S-polygons plotted to scale. It is believed that the reader will be able from the foregoing to establish the S-polygons

for the standard channels so easily as to make it inadvisable to take up the space to show them here. For I-beams no computation is needed beyond what is given in the steel handbooks. Thus would be covered the most important purlin shapes. The writer has never established the S-polygons for angles, but rests content in that they are relatively of minor importance.

TABLE 2.—Apex Co-ordinates for the S-polygons of the American Standard Z-Bars, Expressed in Inches.[3]

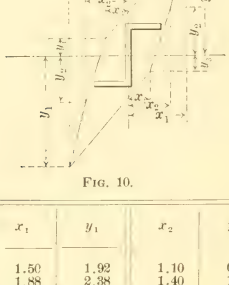

Fig. 10.

Section.	x_1	y_1	x_2	y_2	x_3	y_3
3 × 2½ × ¼	1.50	1.92	1.10	0.88	0.56	0.20
3 × 2½ × 5/16	1.88	2.38	1.40	1.11	0.72	0.23
3 × 2½ × ⅜	2.04	2.57	1.57	1.22	0.81	0.32
3 × 2¼ × 7/16	2.38	2.98	1.88	1.44	0.98	0.24
3 × 2½ × ½	2.45	3.06	1.99	1.51	1.05	0.32
3 × 2¼ × 9/16	2.76	3.43	2.30	1.71	1.22	0.23
4 × 3 × ¼	2.02	3.14	1.44	1.37	0.74	0.42
4 × 3½ × 5/16	2.54	3.91	1.84	1.74	0.95	0.48
4 × 3 × ⅜	3.06	4.67	2.26	2.10	1.18	0.53
4 × 3 × 7/16	3.13	4.83	2.37	2.20	1.25	0.50
4 × 3½ × ½	3.60	5.50	2.77	2.54	1.47	0.53
4 × 3 × 9/16	4.06	6.18	3.19	2.88	1.71	0.56
4 × 3 × ⅝	3.94	6.05	3.18	2.86	1.73	0.50
4 × 3½ × 11/16	4.35	6.65	3.58	3.18	1.97	0.51
4 × 3 × ¾	4.77	7.26	4.00	3.50	2.22	0.52
5 × 3 × 5/16	4.80	5.34	2.00	2.26	1.04	0.80
5 × 3½ × ⅜	3.38	6.39	2.45	2.74	1.29	0.90
5 × 3 × 7/16	3.97	7.44	2.92	3.22	1.55	0.98
5 × 3 × ½	4.02	7.68	3.02	3.35	1.63	0.94
5 × 3½ × 9/16	4.55	8.62	3.47	3.80	1.90	0.99
5 × 3 × ⅝	5.09	9.57	3.94	4.26	2.17	1.03
5 × 3 × 11/16	4.94	9.47	3.91	4.25	2.20	0.94
5 × 3½ × ¾	5.42	10.34	4.37	4.67	2.48	0.97
5 × 3 × 13/16	5.91	11.20	4.84	5.10	2.78	0.99
6 × 3 × ⅜	3.85	8.44	2.75	3.48	1.46	1.36
6 × 3½ × 7/16	4.52	9.83	3.27	4.10	1.75	1.50
6 × 3 × ½	5.20	11.22	3.81	4.72	2.06	1.62
6 × 3 × 9/16	5.24	11.55	3.91	4.88	2.15	1.57
6 × 3½ × ⅝	5.87	12.82	4.44	5.47	2.47	1.65
6 × 3 × 11/16	6.50	14.10	4.98	6.07	2.80	1.71
6 × 3 × ¾	6.32	14.04	4.94	6.06	2.83	1.60
6 × 3½ × 13/16	6.89	15.22	5.47	6.62	3.17	1.64
6 × 3 × ⅞	7.48	16.40	6.02	7.18	3.52	1.67

PLATE IX.
TRANS. AM. SOC. CIV. ENGRS.
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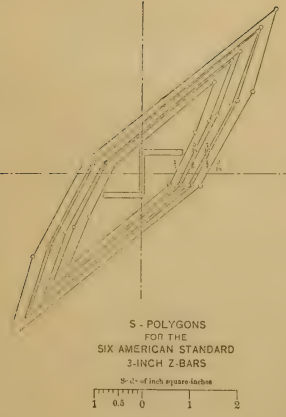
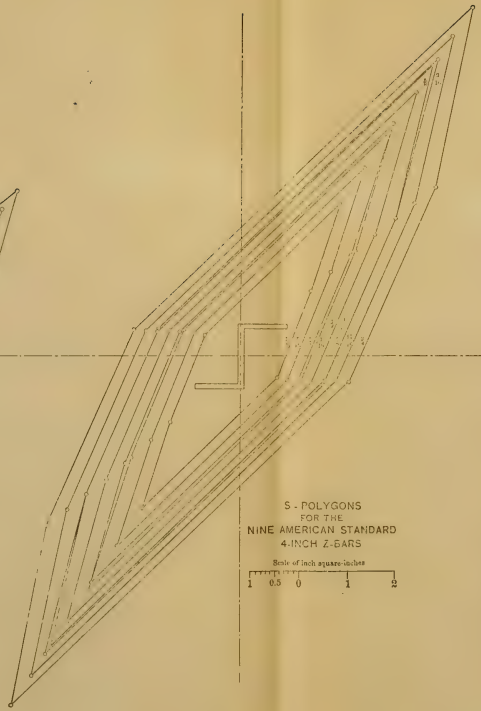
JOHNSON ON
GENERAL FLEXURE.



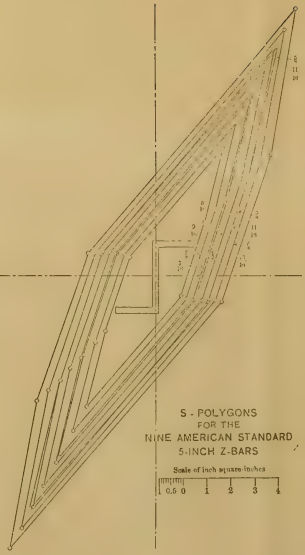
S - POLYGONS
FOR THE
SIX AMERICAN STANDARD
3-INCH Z-BARS

Scale of inch square-inches
1 0.5 0 1 2



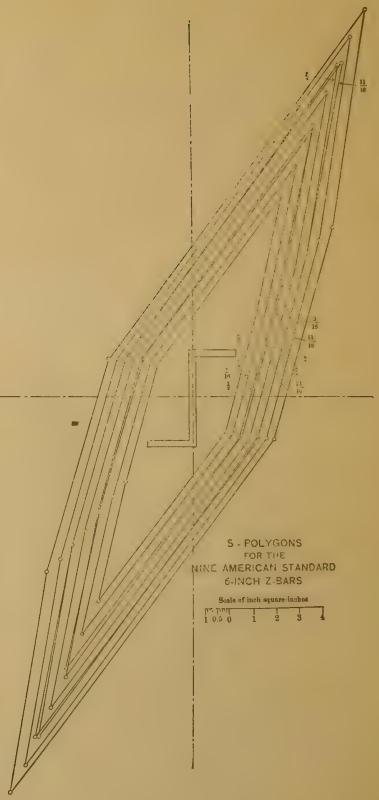
S - POLYGONS
FOR THE
NINE AMERICAN STANDARD
4-INCH Z-BARS

Scale of inch square-inches
1 0.5 0 1 2



S - POLYGONS
FOR THE
NINE AMERICAN STANDARD
5-INCH Z-BARS

Scale of inch square-inches
0.5 0 1 2 3 4



S - POLYGONS
FOR THE
NINE AMERICAN STANDARD
6-INCH Z-BARS

Scale of inch square-inches
1 0.5 0 1 2 3 4

APPENDIX.

11. The Equation and Properties of the Neutral Axis.—Setting $J = 0$ in Equation 10, and rearranging, the equation of the neutral axis is found to be

$$y = \frac{x_k I_x - y_k J}{x_k J - y_k I_y} x + \frac{I_k I_y - J^2}{A(x_k J - y_k I_y)} \quad \ldots\ldots\ldots\ldots (17)$$

the coefficient of x being its slope, and the constant term its intercept, b, on the axis of Y. These values might also have been obtained as tan. α, from Equation 7, and r_o sec. α, from Equations 5 and 6.

Inspection of Equation 17 will reveal a number of important facts.

In the extreme case, when K is at G, y_k and x_k vanish, tan. α becomes indeterminate, and b becomes infinite. That is, if N be applied at the center of gravity of a section, the neutral axis will be at an infinite distance, and with no determinate direction. The f is then constant throughout the section, as might have been foreseen. This is the case of pure normal stress (compression or tension).

For the other extreme, where K is at infinity (the condition for pure flexure), either or both of x_k and y_k become infinite, tan. α is determinate in any given problem (Equation 7) and b vanishes. That is, in pure flexure the neutral axis passes through the center of gravity of the section at an inclination depending upon λ.

For intermediate positions of K, the neutral axis has a determinate slope, and is at a finite distance from G, dependent upon the location of K, and the greater the distance, K G, the nearer to G will be the neutral axis, and *vice versa*.

It can be shown that the quantity, $I_x I_y - J^2$ is never negative. With the aid of this fact, and by finding the intercept of a line through K parallel to n n, it will be seen that K and n n always lie upon opposite sides of G U, as might have been foreseen.

Since tan. α depends solely upon the ratio of y_k and x_k, and not at all upon their actual magnitudes, it is evident that, for all K's on a line through G, the neutral axes will be parallel, and conversely.

Further, the intersection of neutral axes for any two K's upon any straight line, $y = l\,x + b$ will be found to have for its coordinates

$$\left.\begin{aligned} x &= -\frac{J - l\,I_y}{A\,b} \\ y &= \frac{I_x - l\,J}{A\,b} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (18)$$

quantities dependent only upon the constants of the given line and of the section. Hence it appears that for K's moving on any straight

line the neutral axis rotates about a fixed point, given by Equation 18, and conversely. If the line passes through G, b vanishes, x and y become infinite, and the statement of the preceding paragraph is confirmed.

12. The Kernel of a Section.—The kernel of a section is the area bounded by the locus of the K's corresponding to a series of neutral axes touching the periphery of the section but never crossing the section. It could be located with the help of Equations 7 and 18 without difficulty. Its main interest lies in its defining an area within which a K must fall in order that the unit-stress may be of the same sign throughout the section. It is interesting, too, in that its radii vectores are lengths which, for any λ, need only to be multiplied by the area of the section to give S' and S'' for that λ. These lengths would have to be called + if on the opposite side of K from G, and *vice versa*. The kernel is then a figure with sides respectively parallel to the S-polygon, but on opposite sides of G, but unlike the S-polygon in being geometrically an actual surface and to the same scale as the section.

The kernel will be established if the K's be found for the neutral axes coinciding with the sides of the circumscribing polygon of the figure. For each such side considered as a neutral axis there will be a K which will be an apex of the kernel. In the light of the preceding paragraph, the co-ordinates of the kernel apex corresponding to any such side, AB, may be easily written by reversing the signs in Equation 16 and by dividing the second member by A. Accordingly,

$$\left.\begin{aligned} x_{ab} &= -\frac{(x_a - x_b)J - (y_a - y_b)I_y}{A(x_a y_b - x_b y_a)} \\ y_{ab} &= -\frac{x_a - x_b)I_x - (y_a - y_b)J}{A(x_a y_b - x_b y_a)} \end{aligned}\right\} \quad \ldots\ldots\ldots (19)$$

are co-ordinates of a kernel apex corresponding to AB. If AB fall out parallel to the X-axis, these expressions become

$$\left.\begin{aligned} x_{ab} &= -\frac{J}{A\,y_a} \\ y_{ab} &= -\frac{I_x}{A\,y_a} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (19')$$

and, if parallel to the Y-axis, they become,

$$\left.\begin{aligned} x_{ab} &= -\frac{I_y}{A\,x_a} \\ y_{ab} &= -\frac{J}{A\,x_a} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (19'')$$

expressions paralleling Equations 16′ and 16″.

The kernel and the circumscribing polygon of the section are related to each other in such a way that if the K travel along the

one, the corresponding neutral axes will roll around the other; meaning, by that, that they will coincide with side after side of the other polygon, and, pivoting about the apices of the other polygon, will assume all intermediate positions possible without crossing the surface of the polygon.

The continental writers have given much attention to the kernel, but its practical usefulness is so slight, compared with its close kin, the *S*-polygon, that further description of it need not be given here. The preceding section is here given primarily in preparation for the following section.

13.—The S-Polygon and W-Fläche Compared.—The *S*-polygon arrived at in the foregoing was the result of an independent attempt to establish the *W*-Fläche of Professor R. Land, of Constantinople.

The only difference between the two lies in the fact that the sides of the two polygons for a given m lie on opposite sides of G, making the *W*-Fläche just like the kernel with all its dimensions multiplied by A. The writer believes that the maintenance of this distinction, even at the expense of losing the closest possible similarity with the kernel, is worth while. The relations with the kernel will be of little importance in practice, and it seems natural, and hence conducive to accuracy in computation, to call the radius vector positive rather than negative when measured on the same side of G as the point the stress of which it determines. The radius vector will then come to be a kind of pointer starting from the natural origin, G, toward the portion of the cross-section to which it belongs. This is a valuable safeguard against confusion, especially as the diagram is one the use of which is within the comprehension of computers of very moderate experience. Finally, for all sections symmetrical about G, such as I's and Z's, the *W*-Fläche and *S*-polygon would be indistinguishable except in the lettering. Only for sections like C's, and T's would the difference appear.

14.—Closing Remarks on the S-Polygon.—The *S*-polygon is a graphic exhibit of the values of S leading to the extreme unit stresses in the section for all possible inclinations of the plane of loads. It is subject to the advantages and disadvantages of graphical work. It is of the greatest possible simplicity in use, but the measurement of the radii vectores to a high degree of precision may be difficult. A degree of precision quite sufficient for structural designing, however, is easily obtained. Extreme accuracy is rarely justified in practice, considering among other things the uncertainty as to the actual magnitude and inclination of the loads, the inevitable variations of the rolled shape from the nominal section, inaccuracies in construction and erection, and the doubt whether the principle of linear distribution of stress is rigorously applicable. Such considerations make it certainly justifiable to ignore the

minute effect of the rounded fillets and corners of the familiar shapes.

If, for any reason, the computed value of S be required, it can easily be found by substituting the proper x, y, and λ in Equation 11. In fact, a numerical table could thus be computed for as small variations in λ as desired, which would parallel the graphic S-polygon. Such tables have been published in Germany for some of the German standard rolled sections.

Finally, the S-polygon should be observed to be bounded by lines the radii vectores of which for all values of λ have a significance. Radii vectores for points in these lines produced measure values for S which, upon insertion in Equations 13 and 14 will give correct values for the particular point, m, to which the S-line belongs, and the particular λ. The portions of an S-line between two successive apices of the S-polygon are of special importance simply because they mark out the limits of λ's for which an extreme fiber stress will result from the corresponding low values of S.

In closing, the writer wishes to express his obligations to Bruce Borland, Jun. Am. Soc. C. E., who performed most efficiently the exacting task of computing Table 2; and Messrs. A. E. Norton, S. B., J. R. Nichols, and H. W. Telford, S. B., who have assisted in various ways in the preparation of this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS

Paper No. 1021.

THE PANAMA CANAL.*

By A. G. Menocal, M. Am. Soc. C. E.†

With Discussion by Messrs. George B. Francis, Theodore Paschke, Clemens Herschel and A. G. Menocal.

The most difficult engineering problem involved in the construction of a canal at Panama is the control of River Chagres. It enters as an important factor in the design of either a lock or a sea-level canal, and the divergence of opinions, among engineers called upon to decide as to the type of canal best adapted to meet the physical conditions prevailing on the Isthmus, can be traced to the difficulties connected with that river, both as regards the control of the floods, in all types of canal, and the provisions for an ample water supply for operating it during the dry season, in the case of a canal with locks.

It is evident that a lock canal is the most economical type, both in cost and time of construction, and that the sea-level proposition is born either of sentiment or of a belief that by its adoption the difficulties connected with the river can best be overcome. It is well known that a sea-level canal pertaining to the nature of a strait is not possible at Panama. The tidal fluctuation of 20 ft. at the Pacific terminus, while the Atlantic end is practically tide-

* Presented at the meeting of April 4th, 1906.

† Civil Engineer, U. S. N., *Retired.*

less, makes imperative the introduction of a tide lock at Panama, by which ships can be locked up or down, into or from the canal, depending on the stage of the sea level at the time of taking or leaving the waterway. That tidal lock will limit the number of vessels passing through the canal just as much as a series of locks in a lock canal. Some time will be spent in passing each additional lock introduced, and this should not exceed 30 minutes at each lockage, so that the additional time consumed in passing through a canal with six locks would not be more than 3 hours; an insignificant delay for a ship which has saved thousands of miles by taking the canal route.

Considering, therefore, the cost and time saved in constructing a lock canal, as compared with one at sea level, as well as the elimination of the uncertainties and engineering difficulties connected with the latter, it seems that the former type should be adopted, provided the River Chagres can be effectually controlled, an ample water supply provided, and difficult engineering problems avoided.

The object of this paper is to submit for discussion by the Society a modification of the canal route, recommended by the Isthmian Canal Commission of 1899-1901, for a lock canal, by which the River Chagres may be kept under absolute control, its channel being left free to carry off the floods, an abundant supply of water close at hand secured for the operations of the canal, all doubtful problems eliminated, without increasing the cost estimated by the Commission, and with a saving of time in the execution of the work. The change in location recommended begins at Kilometer 46, just south of where the present location meets the Chagres River. On a prolongation of the tangent ending at this point it crosses the River Chagres in the vicinity of Gamboa, and thence, upon the line located by the United States Surveying Expedition of 1875, the canal is kept north of the river, meeting the present location again at Kilometer 9 and coinciding with it to the Harbor of Colon. (Plate X.)

The crossing of the Chagres is proposed to be accomplished by means of a combined dam, viaduct, and controlling works. The dam impounds the river at a maximum elevation of 111 ft. above mean sea level and a minimum of 106 ft. at the end of the dry season.

PLATE X.
TRANS. AM. SOC. CIV. ENGRS.
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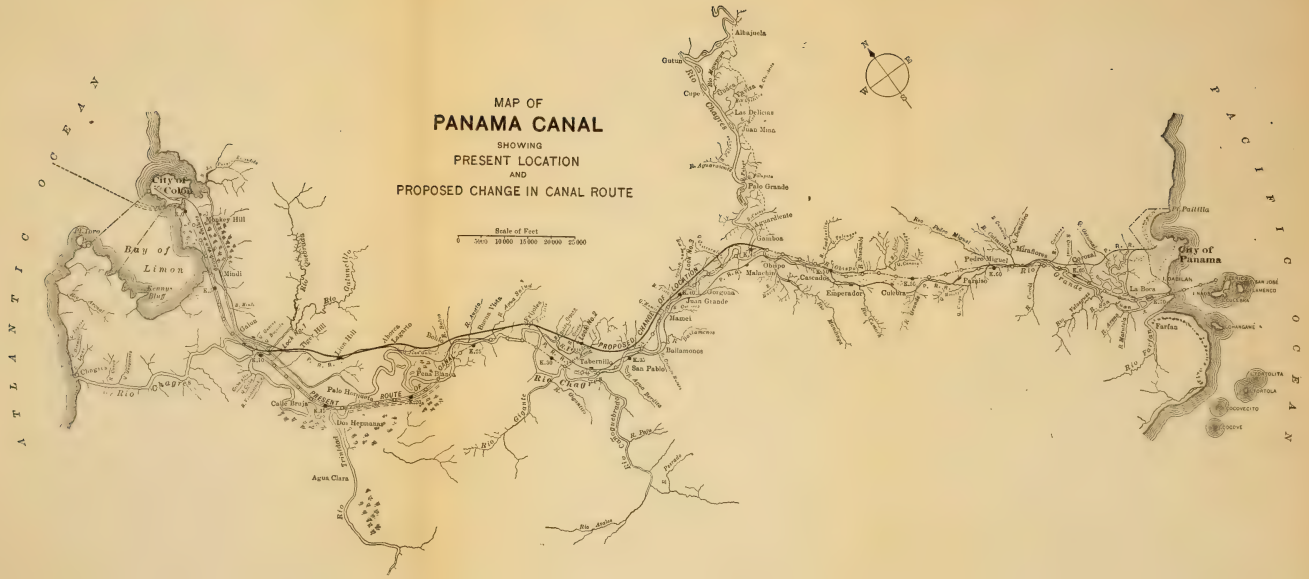
MENOCAL ON
THE PANAMA CANAL.


MAP OF
PANAMA CANAL
SHOWING
PRESENT LOCATION
AND
PROPOSED CHANGE IN CANAL ROUTE
Scale of Feet
0 5000 10000 15000 20000 25000
ATLANTIC OCEAN
PACIFIC OCEAN
City of Colon
Bay of Limon
Mindi
Monkey Hill
Gatun
Alhajuela
Cipe
Las Delicias
Juan Mina
Palo Grande
Gamboa
Obispo
Malaguito
Emperador
Culebra
Pedro Miguel
Miraflores
Paraiso
City of Panama
Paitilla
La Boca
Balboa
Corozal
Juan Grande
Mamei
Ballamonos
San Pablo
Tabernilla
Palo Hermoso
Old Hermoso
Culebra Cut
Agua Clara
Rio Chagres
Rio Grande
Rio Obispo
Rio Pedro Miguel
Rio Paraiso
Rio Grande
Rio Abajo
Rio Arena
Rio Caimito
Rio Trinidad
Rio Gigante
Rio Aguacate
Rio Cocoli
Rio Matasnillo
Rio Farfan
Rio Abajo
Rio Pacora
Rio Tapia
Rio Chilibre

The canal crosses the river in a viaduct, which is also part of the dam, with a summit elevation of 96 ft., the bottom width being 150 ft., and the depth of water 35 ft. As the minimum elevation of the lake formed by the dam is estimated to be at no time lower than 106, the canal can be supplied with the water needed for its operation by short pipes passing through the dam and discharging under the water surface. The water supply is thus kept under perfect control, to be drawn as needed, and the canal can be maintained at a uniform level.

The river controlling works consist of seven compartments or gate-wells (see Plates XII and XIII) and twenty-one 10-ft. metal-lined sluices passing through the structure. These sluices are provided with gates, three in each compartment, operated from the upper platform, by which the river flow can be completely shut off, which may be desirable in the dry season when the river flow may, at times, be less than that required for operating the canal; or the gates may be partially or entirely opened to allow the free passage of floods. With the water in the lake at an elevation of 111 and the lower river in flood, 30 ft. above low water at the viaduct, the twenty-one sluices, under such extreme conditions, will be capable of passing 78 000 cu. ft. per sec., which is far in excess of any known flood discharge at Gamboa. Gate-wells are provided with sill walls rising 10 ft. above low water in the river, intended to arrest any heavy silt or stones rolling on the river bottom. Above the sill walls, the intakes are provided with grooves for the insertion of movable gates, which can be lowered into place or taken up by derricks in the upper platform. With these gates the intakes can be closed and the water drained from a well for examination or repair when needed.

The intakes are protected by heavy iron gratings with openings 2 ft. square to intercept snags, or other floating debris brought down by floods, which might obstruct the sluices or impede the operation of the gates. The water for operating the canal is taken from below the surface of the lake, after passing through the grating, and should be practically free from floating debris. It is discharged into the canal below the surface, so as to cause the least disturbance, or it may be made to discharge through conduits under the bottom of the canal. The upper platform, formed by the dams

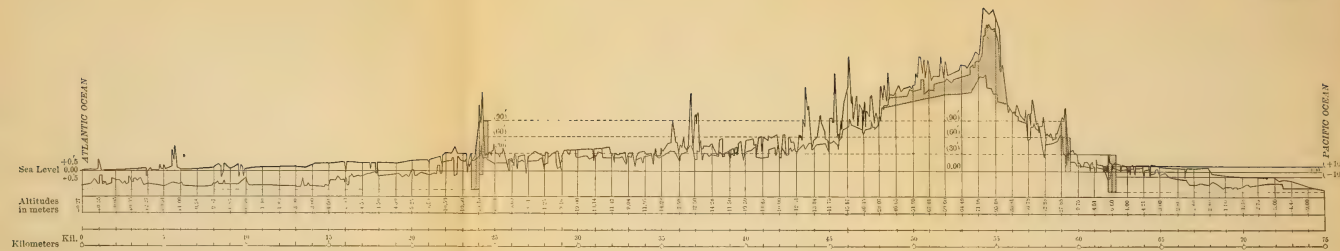
and intake walls, 32 ft. wide, can be roofed for the protection of the gate-operating machinery and the employees manipulating it. The control of the floods and of the water supply can thus be concentrated in one structure.

The combined structure, of reinforced concrete, 232 ft. wide and resting on hard rock, with an elevation of only 118 ft. above the foundations, possesses superabundance of strength and all the conditions of stability and durability essential in a work of this kind. It is believed that it solves effectually the difficult problems connected with the Chagres River, and removes the risks and uncertainties involved in the construction and permanency of dams designed to rest on doubtful foundations. It simplifies the building of the canal, reducing it to a comparatively ordinary engineering work of construction, admitting of an estimate of the cost and time of execution not to be upset by river floods.

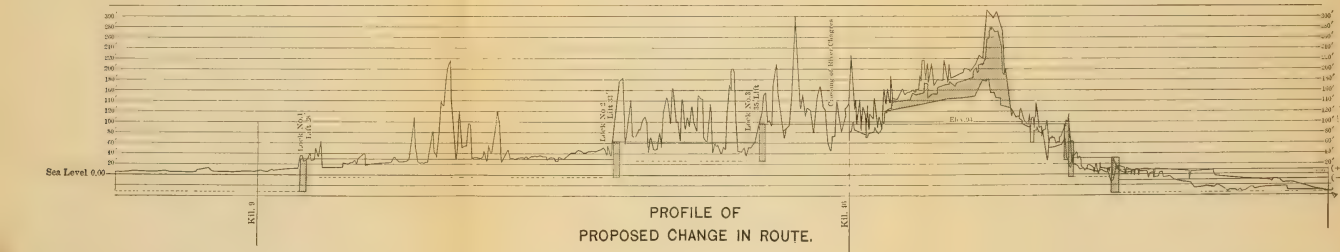
The rock surface under the proposed structure has been plotted from data obtained recently by numerous borings made by the engineers of the Isthmian Canal Commission in the vicinity of Gamboa while exploring for the location of a high dam.

The line proposed from Kilometer 9 to Kilometer 46 of the present canal location was surveyed by the United States Government's Surveying Expedition of 1875 under the direction of the writer as Chief Engineer. It is, however, but a trial location which a limited appropriation did not permit to be surveyed in detail. A final location, after careful development of the topography, will, doubtless, bring about important improvements tending to reduce the excavation and the amount of curvature. It is perfectly practical as now laid down, is about a mile shorter than the present canal location between Kilometers 9 and 46, and involves no engineering difficulties of execution.

With good rock foundations and a wide cross-section of the river at the point of crossing, the viaduct-dam can be constructed with the use of coffer-dams in that portion of the work resting on foundations below water level. The summit level of the canal has been placed at Elevation 96, not imposed by the conditions of the problem, but because that is believed to be the most economical elevation. It is evident that, by lowering the sluices, either by reducing their diameter, increasing their number, if necessary, or



PROFILE OF
PRESENT ROUTE OF CANAL.



PROFILE OF
PROPOSED CHANGE IN ROUTE.

by giving them the shape of inverted siphons, the canal upper level can be lowered several feet, but such modification is not regarded favorably. It would reduce somewhat the lift of the upper locks, which, as herein suggested, is moderate, but at an unwarranted additional cost of excavation.

An alternative plan, which has received consideration, consists in bringing the canal into the lake created by the dam and then crossing the river through the lake instead of the viaduct. By such a change of plan, the proposed works would be reduced to the dam and controlling works, and the cost thereby considerably diminished. The writer is of the opinion that such a modification of the scheme is not desirable. The saving in the cost of the structure would probably be more than offset by an increase in excavation; and cross-currents in the lake, fluctuations of level in the canal, and floating objects brought down by river floods would be serious obstacles to navigation, from which the viaduct plan is entirely free.

Without a contour map of the river basin above Gamboa, the writer is unable to ascertain the superficial area of the lake created by the dam. With a surface elevation of 111 ft. at the dam, the lake would extend to beyond Alhajuela, where the low-water level will be raised about 8 ft. Considering the broad expansions of the river basin and the re-entering valleys of its tributaries, it is contended that the lake will have an area large enough to receive the greatest floods, without violent fluctuations of level, while the whole flow is being carried off by the sluices, and will have ample storage capacity for an abundant water supply for operating the canal in the dry season within the range of Elevations 106 and 111. No reservoir is needed for the storage of floods, and the precise elevations within which the water supply can be assured is a matter of detail which does not alter the essential features of the proposed scheme.

Pending a resurvey of the new location proposed between Kilometers 9 and 46 (which will doubtless result in material improvement in alignment and elevation), and the borings necessary for a classification of the material to be removed, it is not possible to obtain a close estimate of cost of the modified plans.

However, taking for comparison the estimated cost of the

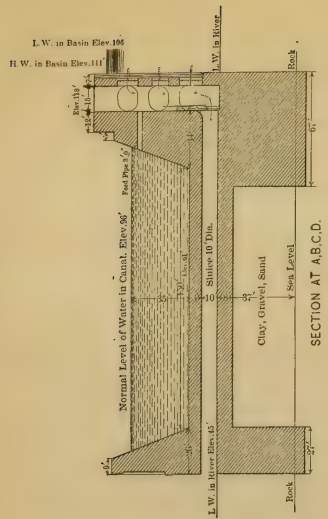
Isthmian Canal Commission's plan for a lock canal with a fluctuating summit level between a maximum of 92.5 ft. and a minimum of 82 ft. above mean sea level, it is possible to arrive at the conclusion that the cost of the modified plan proposed will not exceed the estimate of the Commission, and may fall considerably below it. In any case, it is claimed that if the time of construction can be lessened, and the permanency of the canal at a reduced cost of maintenance can be assured, this would be worth several millions.

The works of the Chagres crossing are estimated to cost $5 275 000. Upon the new route proposed about 52 000 000 cu. yd. will have to be removed from the Harbor of Colon to Kilometer 46, of which 10 000 000 cu. yd. are contained in the sea-level section from the Harbor to Kilometer 9, where it joins the new line. The latter quantity is probably material which can be dredged, the other 42 000 000 cu. yd. being dry soil with some rock in the high ridges crossed.

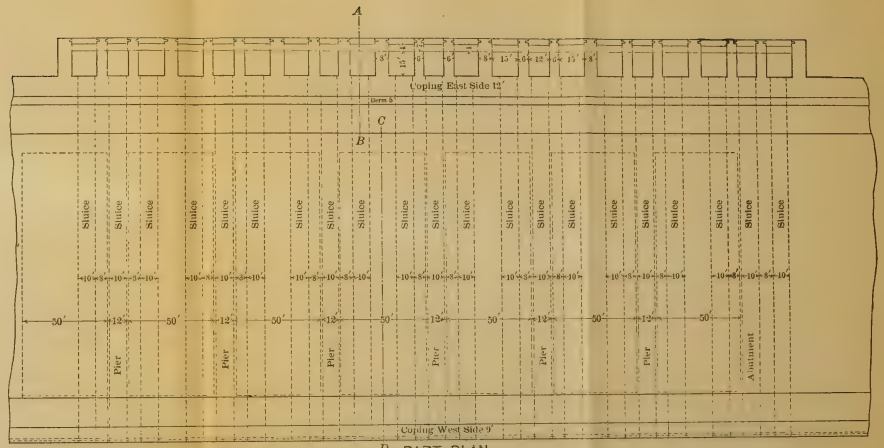
To offset the cost of these works, the following amounts are taken from the estimate of the Commission for work between Colon and Kilometer 45, which would be eliminated by the adoption of the modified plan.

Harbor of Colon to Bohio Locks, including levees	$11 099 839
Lake Bohio	2 952 154
Obispo Gates	295 434
Bohio Dam	6 369 640
Gigante Spillway	2 448 076
Pena Blanca Outlet	1 999 982
Diversion of the Panama Railroad	1 267 500
Total	$26 432 625

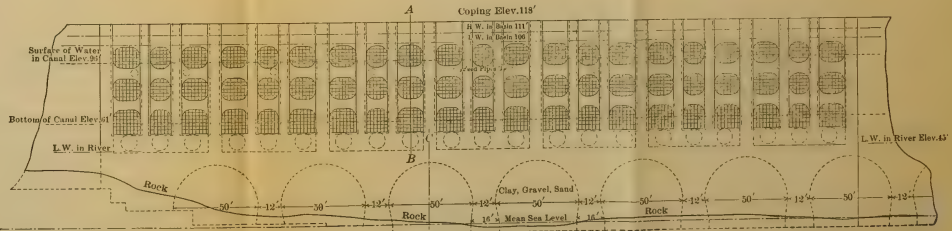
In addition to this total, there would also be a saving of 14 ft. depth in the Culebra Cut, by raising the bottom of the canal from Elevation 47 to Elevation 61, which may be estimated to contain about 20 000 000 cu. yd. The item, $11 567 275, in the Commission's estimate for the two high-lift locks at Bohio will probably balance the cost of the three locks of about the same aggregate lift proposed on the new route now suggested.



SECTION AT A,B,C,D.



PART PLAN



PART ELEVATION-EAST SIDE

PANAMA CANAL

CROSSING OF CHAGRES RIVER

VIADUCT-DAM-CONTROLLING WORKS.

As the basin of the Chagres below Gamboa is not to be flooded by the creation of a lake, the diversion of the Panama Railroad becomes unnecessary. The road can cross the canal by drawbridges, or under it by short tunnels where necessary, and, eventually it can be built on the berm of the canal if so desired.

DISCUSSION.

Mr. Francis. GEORGE B. FRANCIS, M. AM. SOC. C. E.—Among the arguments in favor of a canal of least first cost, viz., a lock canal, the speaker has not seen any of the following kind, and therefore introduces it as a matter which may be of interest.

No man possesses such a prophetic vision that he can forecast (beyond a very limited number of years) what the future will bring forth. The future tendency, however, can be judged by reasoning from the experience of the past.

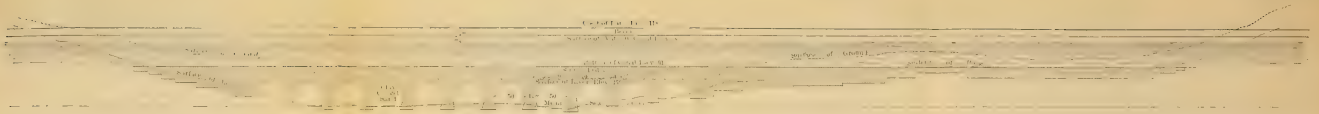
In all construction pertaining to transportation by roads, railroads, and canals, a limited number of years has brought about, from various causes, large and important changes. Sometimes routes and construction works have been entirely abandoned. In other cases, the capacity of the work has been many times enlarged. Again, the amount of funds at first available has seemed ridiculously small after traffic had developed to such an extent that adequate construction could be carried out.

At the end of fifty years, the generation then existing has often ridiculed the inadequate conceptions of the original constructors. Knowing from experience the incapacity of mankind to foresee the requirements of the future, beyond a reasonable period, is it wise to look too far ahead, or to expend much more money than will produce a reasonable waterway, within a reasonable time, and at reasonable first cost?

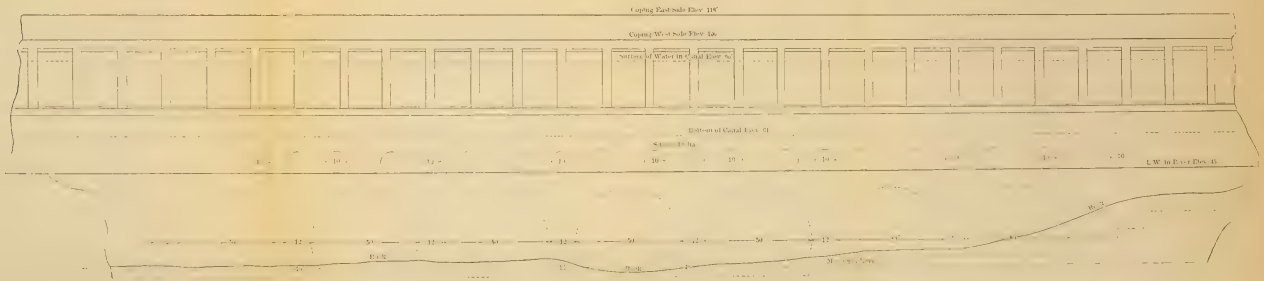
Is it good judgment to be concerned about the way in which this canal will be altered from a lock to a sea-level canal, in the future, or to spend any money to attain that object?

In the absence of knowledge as to the amount of traffic or the possible alteration and increase in the size and draft of vessels, is it wise to prepare for extremes? On one hand, the lock canal may be adequate for many years; on the other hand, it may be entirely inadequate in a short time.

When the time comes for enlargement, engineers will have new ideas, and, instead of enlarging or lowering the canal in its original location, it may be that a new location, perhaps relatively close to the present one, perhaps as far away as Nicaragua, will be thought more desirable; perhaps two canals, at just such a great distance apart, will be considered better than a larger one in one locality. Perhaps two canals adjoining each other, each taking traffic in one direction only, will be preferred. Even though locks are made so that the canal may be changed to sea-level, the traffic may be so great, when the time to make the change comes, that it will be altogether impracticable, on account of the serious interruption to traffic, or the necessity for temporary abandonment of traffic altogether.



LONGITUDINAL SECTION THROUGH AXIS OF CANAL


Coping East side Elev 116'
Coping West side Elev 116
Surface of Water in Canal Elev 85
Bottom of Canal Elev 41
L.W. in River Elev 15
Rock
Mud

PART ELEVATION—WEST SIDE

A great number of examples could be cited in support of such conditions, not only in transportation lines, but in public works like fortifications, river improvements, and many minor kinds of construction. Mr. Francis.

Fifty or one hundred years from now, the wealth and energy of the United States will be beyond the possible conception of any person now living, and the expenditure of $100 000 000, which is a large sum to-day, will seem to be relatively small to the statesman or engineer of the future.

Looking at the whole problem of the Panama Canal from this point of view, is it not good judgment to build now a good, substantial lock canal for substantially the needs within view, or, in the event of a decision to build either a sea-level or a lock canal, to expend no appreciable amount of money thereon in preparation for changes or enlargements which may be remote, and may be made in a manner entirely different from that now conceived?

THEODORE PASCHKE, M. AM. SOC. C. E.—The special feature of Mr. Menocal's paper is an argument in favor of a lock canal, culminating in the somewhat novel proposition of the combined viaduct-dam at Gamboa. Mr. Paschke.

However interesting this feature may be to the lock-canal partisans, it will be well to defer its full consideration until a necessity therefor has arisen, and until it has been fully decided that a lock canal shall be built. This opportunity is taken to express the hope that that time may never come.

To the speaker, this paper seems to be a very timely invitation to the engineers of the country to express their opinions on the all-important question which at present is before the United States Government for decision; and it behooves this Society to speak freely on the question as to the type of canal which should be adopted.

It is with this in view that the speaker ventures a few remarks on this subject.

Mr. Menocal says:

"It is evident that a lock canal is the most economical type, both in cost and time of construction, and that the sea-level proposition is born * * * of sentiment," etc.

The question of comparative economy between the two types is fully covered, and may best be left to be answered by the testimony given before the Senate Committee by J. F. Wallace, Past-President, Am. Soc. C. E., the late Chief Engineer of the Canal.

As to the matter of the sea-level proposition being born of sentiment, why, the speaker, for one, on the sea-level side, admits this at once, and without hesitation. All achievements in the march of civilization, of improvement, are born of sentiment. Was not the voyage across the Atlantic of that most intrepid of all navigators,

Mr. Paschke. Christopher Columbus, which resulted in the discovery of this continent, born of sentiment? And how about the Declaration of Independence? And still later, the preservation of the Union? Were these not born of sentiment? Here is a whole chain of grand, noble sentiments, one after the other, growing out of the primary sentiment entertained by the Genoese ancient mariner, culminating finally in the reasonable expectation of the realization of his dream of more than four hundred years ago. Why, then, should sentiment detract from the sea-level proposition?

The speaker is inclined to remind the author that we are at Panama, not at Nicaragua. If he were at the latter place, he would be justified in sweeping aside the sea-level proposition as born of sentimentality. But, at Panama the sentiment of the sea-level proposition becomes tangible and within the range of practicable execution. Yea, more, the sentiment becomes conviction that the sea-level proposition is the best and the only proposition worthy of consideration.

All will admit that the underlying sentiment of an ideal type for a ship canal across the Isthmus is a free and unobstructed passage of ships, and all, even the most fervent enthusiasts of the lock proposition, will admit that a lock in a ship canal is an obstacle, a hindrance, to such free and unobstructed passage, notwithstanding the labored paradoxical arguments advanced in the minority report of the consulting engineers. Now, reducing the question of superiority of type to a simple arithmetical proposition, there is the lock-canal proposition with four or more obstacles in the way of a free and unobstructed passage of ships, as against the sea-level proposition with only one lock, and that one open for one-third of the time. There is the underlying sentiment of the sea-level proposition.

There is no desire on the speaker's part to make light of the engineering difficulties in the way of a realization of the proposition for a sea-level canal, but he has faith in the resourcefulness, in the genius of American engineering talent to solve all problems in connection with the proposition completely and successfully, when confronted with the mandate of the nation.

Mr. Herschel. CLEMENS HERSCHEL, M. AM. SOC. C. E. (by letter).—Proof appears to be lacking that, as assumed in the paper:

"It is well known that a sea-level canal pertaining to the nature of a strait is not possible at Panama. The tidal fluctuation of 20 ft. at the Pacific terminus, while the Atlantic end is practically tideless, makes imperative the introduction of a tide lock at Panama, by which ships can be locked up or down, into or from the canal, depending on the stage of the sea level at the time of taking or leaving the waterway. That tidal lock will limit the number of vessels

passing through the canal just as much as a series of locks in a lock canal." Mr. Herschel.

This is a subject the importance and bearing of which are far beyond any considerations of what is to be built finally at Panama; so that it deserves attention, whether or not the details of the Panama Canal have been settled.

There are in existence at the present day three sea-level canals without locks: The Corinth Canal, the Suez Canal, and the East Bay Neck Ship Canal, in Tasmania.

A study of their history may prevent future assumptions that a tidal variation at one or both ends of such canals, of material length, necessarily calls for locks.

The Corinth Canal.—The sea-level canal across the Isthmus of Corinth was projected by Periander, Tyrant of Corinth, 628 B. C.; also by Demetrius Poliorcetes, one of the successors of Alexander the Great. Work on it was done by the Roman Emperor Nero, and we read that:

"The Egyptians, to whom Demetrius Poliorcetes had confided the studies of the enterprise, declared that the Gulf of Corinth was so much higher than that of Egina, that the latter bay would be in danger of having its islands and shores submerged. The abandonment of Nero's scheme is due, probably, to the same error, and to the superstition of the natives, carefully encouraged by the priests of Corinth, who were fearful of seeing the number of travellers visiting their sanctuary diminished."

Precisely similar objection was made to the construction of the Suez Canal, and is yet before the world, on authority, but unsupported by science or computation, concerning the Panama Canal.

The Corinth Canal was completed about 1896; some 2 500 years after Periander. What the nature of the tides is in the two bays which it joins, and what has been their result in the way of producing currents in this sea-level canal without locks, the writer has been unable to ascertain with precision. It is stated, however, in an appendix to the Report of the Board of Consulting Engineers for the Panama Canal, that among others, the *Lusitania*, a vessel of 42.5 ft. beam, drawing 20 ft., and 320 ft. long, has passed through the canal, and a newspaper account* states that the current in the canal, on a day indicated, was 3 knots per hour.

It may properly be inferred, therefore, that the tidal currents of this canal are not an important factor in its operation, especially not for modern steam, or other power, navigation; nor have the allied dire predictions of the old-time Egyptians, previously noted, come true.

They, and others like them, did this, however: For 2 500 years

* *New York Tribune*, March 18th, 1906.

Mr. Herschel. they hindered and obstructed, and presumably made impossible, the construction of this long-desired work, by reason of fancies entertained with regard to it, and by putting forward these fancies, in the way of the earnest workers who, during all that time, were striving to bring about a needed public work.

The Suez Canal.—It will not be necessary to rehearse all the opposition to the construction of the Suez Sea-level Canal by engineers and others, though a reading of its history will always remain most instructive. It should be read before reading the reports on the Panama Canal.

Said Lord Palmerston, in condemning the scheme as "one which no Englishman with his eyes open would think it desirable to encourage":

"As regards the engineering difficulties, I am aware there is nothing which money and skill cannot overcome, except to stop the tides of the ocean, and to make rivers run up to their sources. But I take leave to affirm, upon pretty good authority, that this plan cannot be accomplished, except at an expense which would preclude its being a remunerative undertaking; and I, therefore, think I am not much out of the way in stating this to be one of the bubble schemes which are often set on foot to induce English capitalists to embark their money upon enterprises which, in the end, will only leave them poorer, whomever else they may make richer."

Robert Stevenson then arose, and said:

"He would not venture to enter upon the political bearings of the subject with respect to the other powers of Europe, but would confine himself merely to the engineering capabilities of the scheme. He had travelled, partly on foot, over the country to which the project applied, and had watched with great interest the progress that had been made by various parties in examining the question. He had first investigated the subject in 1847, in conjunction with M. Paulin Talabot, a French engineer, and M. de Negrelli, an Austrian engineer. At the suggestion of Linant Bey, a French engineer, who had been upwards of twenty years resident in Egypt, and feeling how important was the establishment, if possible, of a communication between the Red Sea and the Mediterranean, he had qualified himself to form an opinion on the subject. * * * * *

"Commercially speaking, he frankly declared it to be an impracticable scheme. What its political import might be he could not say, but as an engineer he would pronounce it to be an undesirable scheme, in a commercial point of view, and that the railway (now nearly completed) would, as far as concerned India and postal arrangements, be more expeditious, more certain, and more economical than even if there were this new Bosphorus between the Red Sea and the Mediterranean."

In common with some other railroad men, Robert Stevenson, though a great engineer, seems to have thought that there was nothing like railroads.

Mr. Herschel.

But, let that pass. This canal, which was to have brought ruin on all who were in any way connected with it, has become a well-established and most profitable piece of property, paying more than $17 000 000 annually to its stock and bond holders, and now has Great Britain for its principal owner, by purchase of shares.

The currents through the Suez Canal, concerning which there were prophecies of dire calamities—some, notably the ablest English engineers, claiming that they would be strong enough to destroy the canal, and must be controlled by locks; while others thought there would not be sufficient current generated to keep the harbors open—have proven to be a benefit rather than an objection. They are fully described in *Annales des Ponts et Chaussées*. 1898, 3e Trimestre.

Briefly stated, there is a Red Sea tide at one end of the Suez Canal, of about 5½ ft.; 2 or 3 ft. more than this during occasional Red Sea storms. That is to say, to be fair about it, the level of the Red Sea varies from a level of about 3 ft. above, to a level of about 3 ft. below, mean level of the sea; while, 15 miles away, the Bitter Lakes hardly vary in level at all. There is thus produced a slope of water surface (between ends) causing for 6 hours a current to the north, alternating with the same slope and a current to the south for another 6 hours, with a period of slack water, or of no current in either direction, in the middle of each 6-hour period, and so on; approximately like a huge teeter, 15 miles long, hinged at one end, in perpetual oscillation. These currents thus produced, on a slope, between end water levels, of 2½ in. to the mile of canal, attain a velocity, at the four daily moments of greatest velocity, of never more than 2.67 ft. per sec. (1.6 knots per hour) in spring tides.

But, be it noted: These currents, of no harm in themselves, are the creatures of but a moment. They begin to die, as they are born; that is to say, their death is fore-ordained with their gradual creation, lasting 3 hours, to take place within the following 3 hours. They begin from a slack-water, or no current, and requiring 3 hours to attain a maximum as given, they immediately slack off again during another 3 hours, until another slack-water is reached, and so on, making four times of slack-water every lunar day. So that it would be as true to state that there is no current in the canal, as it is to state only that the currents have a velocity of 1.6 knots, as is so frequently done.

Be that as it may, the Suez Canal has shown itself to be a navigable channel, and any proposition to put a lock into it, for any reason or purpose, would meet with just scorn. The fancies of engineers and statesmen, held with regard to it before its construction, have been nigh universally forgotten; but it is instructive to have them restated, that similar fancies held at the present day, and even of probable future recurrence, may be recognized as such, and rejected.

Mr. Herschel. *East Bay Neck Ship Canal, Tasmania.*—History appears not to have recorded the dire predictions made before the construction of this little, lockless, sea-level ship canal, situated as far away from common routes of travel as one is apt to voyage. Unquestionably, such predictions, of the currents sawing into halves the globe which we inhabit, or else wrecking vessels which might attempt to stem the Charybdis to be formed, were not wanting. Projects for sea-level canals seem to educe that effect out of the human mind. But, in the *Minutes of Proceedings* of the Institution of Civil Engineers* it is stated that:

"From observations made previously it was calculated that a current would flow in the canal from north to south; it is now found that a current of 2 to 3 miles an hour flows in winter from south to north, and in summer from north to south, with irregular intervals during gales."

Panama Canal.—This is a large subject, and let no one suppose that it is about to be discussed in all its entirety. That would call for the expression of many opinions, which the writer at present is not prepared to give. But, as illustrating the particular engineering and popular idiosyncracy of jumping to the conclusion that a sea-level canal palpably needs a lock, the recent history of this momentous enterprise offers a most instructive lesson for future avoidance. Were it not for future guidance with respect to other sea-level projects, it would at date presumably have little value, and certainly would not have been written, for the construction of the Panama Canal has now been fairly launched on the sea of chance effects; or, possibly, its mode of construction has been settled upon, and is no longer open to discussion.

The first official statement, that a sea-level canal at Panama could not be built without a lock, came in the letter of the Chairman of the Isthmian Canal Commission to the Board of Consulting Engineers, of September 1st, 1905. This is what was said in that letter:

"A disadvantage which the two plans have in common, is that the rapid developments of naval architecture make it difficult to determine the proper dimensions of the lock chambers. * * * * * * * * * * Moreover, it is not possible to dispense with locks entirely. Even with the sea-level canal, a tide lock will be required at the Panama end."

Such instructions to a Board of Consulting Engineers, from one who is not an engineer, presumably were based on some engineer's statement carrying conviction with it. Perhaps these instructions were based on page 224 of the book, "Problems of the Panama Canal," 1905. But truth is mighty, and must prevail, and especially must the future be safeguarded.

* Part IV, 1904-1905, p. 370.

Mr. Herschel.

The following is found on page 224:

"It may be remarked, at the outset, that a construction wholly without locks is impracticable at any expense, since the tidal oscillation of the Pacific, about 20 ft., can only be controlled by a lock near Miraflores. One lock being a necessity, the addition of four others becomes less objectionable."

Where is the proof of any such statement? Beyond stating that a 20-ft. tide exists at one end of the canal (an oscillation of 10 ft. on either side of mean level of the sea is meant), no facts or results of computation are given.

Let us supply these, for, without them, authority, still more, *obiter dicta*, should be viewed with suspicion, in the judging of an engineering project, rather than be allowed to rule.

It has been seen that a slope, between end water surfaces on the Suez Canal, of $2\frac{1}{4}$ in. to the mile, produced a maximum velocity of 2.67 ft. per sec. (1.6 knots); and it will not require profound hydraulic computations to show that, in a Panama Canal, which would have a similar maximum slope of water surface of 3 in. per mile, on 44 or 45 miles of sea-level canal, the engendered velocities will not be inordinately great. But the precise computations were long ago made, by Boussinesq, and other masters of the science of hydraulics, and may be found in the records of the meetings of the French Academy.*

The situation, as found by the able Committee of the French Academy, would be exactly similar to the one described as existing in the Suez Canal: a long water surface, no matter now whether plane, or slightly curved, or how curved, this time 45 miles long, hinged, as it were, at one end, and the other end oscillating some 10 ft. above and below the fixed mean level of the sea, making alternately a slope, between end water surfaces, and currents, to the north, and to the south, about 3 in. to the mile of such maximum slope, and maximum currents of about 4 ft. per sec. ($2\frac{1}{2}$ knots). On rare occasions during the year, brought about by strong gales, this may be exceeded, and there is room for the excess without materially obstructing navigation. Vessels do navigate channels in which the velocity of the currents is 5 and 6 knots and even greater, up to 10 and 12 knots in narrow channels on river rapids. But it is not the writer's purpose to discuss debatable questions, and the precise limit of speed at which currents become of material hindrance to navigation, is such a question.

But there should be no difference of opinion as to the statement that the velocities in the Panama Canal should have been computed years ago, and conclusions based on assumptions with regard to them should have been studiously avoided. It would undoubtedly require

* *Comptes Rendus*, 1887, Vol. 104, p. 1484.

Mr Herschel

the work of one or two skilful hydraulic engineers for a month or more to make such a computation, but what is that to the capabilities of the United States Government, and in a cause of this sort? It is too much, however, to expect from the plain United States citizen as a volunteer offering, so that its absence in this discussion may be pardoned.

It must be remembered that this is no mere case of uniform flow, or of the application solely of a Kutter or a Chezy formula. Equally important and applicable is the law of the propagation of water levels by means of water waves, which controls, even acts as a check on, the ordinary canal flow formulas. A moment's thought will prove this.

In the ordinary canal formulas, an increase of cross-section notably increases the velocity, other things being equal, and so on without limit. If this were true in sea-level canals and channels, the Harlem River would be unfit for navigation, and the East River would be like the Niagara Rapids just up-stream from the Whirlpool. But Nature is much more resourceful than all that. It requires, to be sure, some length of channel to moderate the action of gravity and tidal influences on these large bodies of water, but, with 40 odd miles of canal through which the water can flow, it can readily be understood that the generated currents need at no moment of time exceed 4 ft. per sec. in the Panama Canal cross-section of 1887, as computed by Boussinesq and others, and immaterially more than this for the somewhat greater cross-section now proposed; and, should the canal even be widened into a strait—a somewhat doubtful possible event—the natural effect would soon be to improve the free navigation, not to render it impracticable.

The recent Board of Consulting Engineers received instructions from the Chairman of the Isthmian Canal Commission with regard to lock gates on September 1st. Many engineers, and a host of others, immediately jumped to the conclusion quoted. In magazine articles and newspapers the cry was immediately taken up that "one lock being a necessity, the addition of four others becomes less objectionable," and the world will probably never know what part this delusion, at first apparently only a chance remark, or one made without due reflection, had in determining, in the Board of Consulting Engineers, the type of canal to be constructed at Panama.

As early as September 16th* "one of the Board of Consulting Engineers" was publicly quoted as decided for a lock canal, and, thereafter, the kind of *vox populi* above named had full sway.

This is not saying that, in certain cases, *vox populi* has not merit. But it should be remembered, that it cannot control physical facts, and it has done many erratic things.

Boston Daily Advertiser.

Considerations, such as stated, therefore, should not be allowed to govern the construction of future sea-level canals. There is reason for the assertion that it required persistent effort to wring the following from the majority of a divided, undoubtedly compromising among themselves, perhaps overburdened, board: Mr. Herschel.

"The question of the necessity of a tidal lock at the Panama end of the canal has been raised by engineers of repute, but the limited time available to the Board has not permitted the full consideration of this question, which is desirable. It is probable that in the absence of a tidal lock the tidal currents during extreme spring oscillations would reach 5 miles per hour. While it might be possible to devise facilities which would permit ships of large size to enter or leave the canal during the existence of such currents, the Board has considered it advisable to contemplate and estimate for twin tidal locks located near Sosa Hill, even though the period during which they would be needed would probably be confined to a part of each spring tide.

"The highest recently recorded range of spring tides which the Board has seen (September, 1905) was 19 ft. 9 in. between extreme low and extreme high water, while from 1882 to 1887 the highest amplitude reported was 20.93 ft. With such tides for a brief period at dead low water there would be a differential head of about 10 ft.—that is to say, the water in the canal (45 miles away)* would be 10 ft. above that in the bay, while at extreme high water for a correspondingly short period the level of the water in the bay would be 10 ft. higher than that in the canal (45 miles away).*

"At the period of mean tide there would be no difference of level between the bay and the canal, so that during that period of the tide all the gates of the tidal lock could be open, leaving an unobstructed passage for vessels until the approach of the flood tide rendered it necessary for the gates to be closed until slack water would again be reached, and so on for each succeeding spring tide. During neap tides the range is so small that it will not be found necessary to bring the gates of the lock into use. Consequently, throughout the neap period of each tidal cycle a continuously open and unobstructed passage for traffic will be provided through the tidal locks."

All this errs, it is here argued, on the side of that safety which is the offspring of unfounded fear. In this there would be no especial harm, except that the proposition to be handling huge lock-gates to control currents, is believed to be impracticable. It would also disarrange the normal and balanced natural play of the tides in establishing harmless currents, throughout the length of the canal, from the Atlantic to the Pacific. A lock is also a constriction of the normal area of the body of water of the canal. For the water to pass through the lock will require an undesirable increase of the normal velocity within the lock. In other words, the canal must either be all free, or all lock.

*The words in parentheses have been added.

Mr. Herschel. The sea-level canal could be built without a tide-lock, and of uniform cross-sectional area, without lack of deference to the doubting; for a tide-lock could be added at any time.

But let all that also pass; the point to be emphasized is the expression of the hope that the next project for a sea-level canal will not be afflicted with the unwarranted assumptions of its opponents, as has been that of the Panama Canal, and, apparently, of each of the three lockless sea-level canals now in existence, assumptions which before now have altogether prevented the construction of sea-level canals.

Mr. Menocal. A. G. MENOCAL, M. AM. SOC. C. E. (by letter).—As this paper was prepared before the Board of Consulting Engineers for the Panama Canal had completed its labors and made public its conclusions, the writer had before him for comparison with his plan for a lock canal at Panama the previously accepted project of the Isthmian Canal Commission of 1900-1901. He has been informed since, by the reports of the majority and the minority of the Board, that the former recommends the construction of a sea-level canal at an estimated cost of $247 021 200, with from 12 to 13 years as the construction period, while the latter recommends a lock canal at the 85-ft. level, estimated to cost $140 000 000, and to be completed in 9 years.

After careful consideration of the arguments elaborated in the reports, both for and against the two plans proposed, the writer still claims that the lock canal which he advocates is the best solution of the problem of a canal at Panama. If enlarged in water section so as to conform to the depth of 40 ft., as recommended by the Board, and a bottom width of not less than 200 ft. between the locks and 400 ft. in the sea-level section, its cost may exceed the estimated cost of the 85-ft. plan proposed by the minority, but it is believed that the increased cost, if any, would be a profitable investment in the interest of a better canal. It is less tortuous, more free from currents, from submerged channels and from flood waters and sedimentary deposits, and, above all, its safety is capable of mathematical demonstration, supported by the best and most conservative engineering practice, which cannot be claimed for the controlling features of the minority's project. In the latter plan the safety of the canal is dependent on the efficiency and permanency of an earth dam, 7 700 ft. long, resting for a distance of 2 640 ft. measured at sea level, on two gorges, 208 ft. and 258 ft. deep, respectively, confined by steep, rocky cliffs and filled with alluvial deposit, *viz.*, sand, clay, shells, wood, silt, gravel, etc., promiscuously distributed and arranged by the River Chagres. The stability of this dam, impounding a lake of 118 sq. miles, and 85 ft. deep at the dam, is based on the theory that the gorges referred to are filled with an im-

pervious blanket 200 ft. thick, upon which the embankment is to rest, and that, for that reason, leakage under the dam will be inappreciable. That theory, however, appears to be disproved by the testimony of engineers, supported by the records of the Isthmian Canal Commission, before the Committee on Interoceanic Canals of the United States Senate. It was shown by borings, taken at various places and penetrating to various depths in those gorges, that at all points where the lower end of the pipe was in the alluvial deposit, water flowed out of the upper end of the pipe several feet above the ground, showing, conclusively, that the material in those gorges, far from being impervious, is water-borne, and that water under considerable pressure flows freely through the mass. These unfavorable features of the dam site, together with the uneven settlement that must be expected on account of violent changes in the character of the sustaining soil, especially at the crests of the rocky cliffs, under the weight of from 7 to 8 tons per sq. ft. of embankment, and exclusive of the 85-ft. head of water, seem to point out conditions under which dependence on such a dam for the safety of the enormous interests involved in this canal, may be regarded as a hazardous undertaking. Mr. Menocal.

Reconstructing the proposed plan by lowering the Gatun dam and building another low earth dam across the Chagres at some place above that point, say, at Bohio, as has been suggested, would not, in the writer's opinion, overcome the difficulties, but rather increase them. Water now flows freely through the geological gorges at both Bohio and Gatun, and nothing short of a masonry dam resting on solid rock, or an earth dam with a masonry core, will cut it off. Two low dams would mean two shallow lakes, in lieu of the 85-ft. level lake extending from Gatun to Pedro Miguel, involving more difficulties in handling the flood waters, in providing for the water supply for operating the canal, in removing sedimentary deposits, and in navigation through the lakes.

The risks inherent to the proposed flight of three locks with an aggregate lift of 85 ft., located at Gatun, within six miles of the sea coast, and the disastrous results that would follow a failure of the upper gates, either through accident or injury by an enemy, have been fully brought out in the report of the majority of the Board and in the hearing before the Committee of the Senate, and need not be dwelt upon here.

The majority of the Board has brought out in its report, with considerable emphasis, many interesting arguments tending to show the advantages of a sea-level canal over one with locks, but the writer still adheres to the opinion that a lock canal of suitable dimensions and free from works of doubtful stability will be ample to accommodate all the traffic likely to use the Isthmian transit for

Mr. Menocal. many years with the same facilities and safety to navigation as can be provided by a sea-level canal, except for the additional time consumed in passing through the lift locks, which, for the canal proposed in this paper, should not exceed three hours; and it is believed that no ship-owner would be willing to pay a bonus sufficient to meet his proportional share of the interest and amortization of the additional cost of the work, in order to save that short time in the long passage of his vessel. As the volume of traffic increases, the high-level waterway can be enlarged so as to admit of greater freedom and speed in transit, at very much less cost and in less time than would be involved in similar enlargement of the sea-level canal, by reason of less depths in the excavations and greater facilities for disposing of the spoils. As to liability to accidents at the locks, which is frequently referred to in the report of the Board, it may be said that experience in lock canals, especially in the St. Mary's Falls Canal, where the largest lift locks in the world have for many years passed an enormous annual traffic without serious accidents or interruption of business, shows that such accidents are quite remote, and that by observing proper precaution in handling the vessels they can be reduced to a minimum or avoided altogether. It may be said that the sea-level canal proposed is not free from dangers of that kind, either from difficulties in navigating a large ship through a restricted tortuous channel with rocky sides, with or against currents which have been estimated at 2.5 miles an hour, or from failure of dams or embankments built, partly at least, on alluvial soil and impounding large bodies of water suspended at considerable elevation above, and close to, the canal.

There seems to be a tendency in the minds of many, and the matter received much consideration by the Board of Consulting Engineers, that if a lock canal is built at Panama it should be of such a type as will admit of being converted to the sea-level plan as soon as the demands of commerce require such transformation. While not believing that such a time will ever arrive, because the lock canal can be made to meet all the requirements, it may be proper to state that the plan proposed herein has the additional advantage of being capable of such a change in type, and that when so transformed it will be a better and safer sea-level canal than any yet proposed, for the very good reason that the flood waters of the Chagres above Gamboa would have a free outlet to the sea through the disused lock canal from Gamboa to Gatun, and thence by the canal of derivation partially completed by the French Company, but enlarged and protected by embankments where needed to carry off the additional flow into Mansanillo Bay.

The excavation for a sea-level canal between Gatun and Obispo and the diversion of the tributaries of the Chagres below Gamboa

can be carried on without restrictions imposed by the canal traffic, and, as the Culebra section is cut down, locks can be built south of the viaduct to descend to the new levels created, until the sea level is reached. These locks would be abandoned when the low-level connections are completed, but, as they will be temporary structures shaped out of the solid rock, the cost need not be large. Mr. Menocal.

Considering its enormous cost; the long time required for its completion; the uncertainties surrounding the work; the great expense and delay involved in its enlargement; the difficulties in navigation on account of tortuous, narrow channels, currents, and sedimentary deposits brought down by the rivers discharging into it; the writer is of the opinion that a sea-level canal at Panama should be undertaken only as a last resort, *viz.*, after it has been proved that a safe lock canal cannot be constructed there.

That a tide lock will be needed at the Pacific end of a sea-level canal built at Panama, has been demonstrated by the practical operations of other canals with tidal oscillations at their termini. At the Red Sea end of the Suez Canal the maximum rise and fall of the tide is from 7 to 8 ft. The current produced in the canal by this fluctuation of levels, extends for a distance of 12 miles to the Bitter Lakes, where it disappears. It attains a speed of about 3 miles an hour near the canal entrance. This current does not impede navigation through the canal, but it obstructs it to the extent that ships moving in the direction of the current cannot be stopped; those going in the opposite direction being tied up to allow the others to pass. The rise and fall of the tide at Panama being nearly three times as great as at Suez the current developed in the canal must be proportionally greater, and navigation with or against the current impracticable.

At the point where the Manchester Canal meets the River Mersey, the rise and fall of the tide is about 20 ft., practically the same as at Panama. Mr. Hunter, Chief Engineer of the canal and a member of the Board of Consulting Engineers of the Panama Canal, has declared that, from his observations at Manchester, a tidal lock would be necessary at Panama, and the Board was unanimously of that opinion.

In the case of the Kiel Canal it has been found that the tidal currents through the tide lock, when the gates are open, offer such an impediment to the manipulation of the gates, that the original expectation that they would be kept open a great portion of the time has not been realized, and they are kept closed.

The writer is of the opinion that the expectation that the gates of the tide lock at Panama may be kept open a portion of the time will meet with the disappointment experienced at Kiel. The canal surface being at about mean sea level, it is quite evident that when

Mr. Menocal. the tide in the Bay of Panama reaches that level, either in its rise or fall, it will be running so strong that the current through the locks will interfere seriously with the operation of the gates, and no ship can approach the lock or attempt to pass between the gates with safety. At dead high or dead low water, the surface of the canal will be from 6 to 10 ft. below or above the level of the water in the bay, and with the gates open the current through the lock and canal will render navigation impracticable.

A mathematical demonstration of the curves of tidal currents in the Panama Canal, under the constantly varying conditions of level, winds and other factors affecting that complicated problem, would be very interesting from a technical point of view, but it is not likely to lead to practical results different from those shown in the operations of other canals.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1022.

THE SCRANTON TUNNEL OF THE LACKAWANNA AND WYOMING VALLEY RAILROAD.*

By George B. Francis and W F. Dennis,† Members Am. Soc. C. E.

With Discussion by Messrs. F. Lavis, V. H. Hewes and W. F. Dennis.

During the years 1901, 1902 and 1903, Westinghouse, Church, Kerr and Company constructed a double-track, high-speed, electric railroad, on private right of way, in the heart of the Northern Anthracite coal field, with termini at Wilkes-Barre on the south, and Scranton on the north, a distance of about 20 miles.

The entrance to Scranton was over the hill on the southerly side of the city, with 4% grades, on a temporary line built to permit an early opening of the road.

During 1904 and 1905 a tunnel was constructed through this hill, on a permanent line and at moderate grades. The object of this paper is to describe the construction of this tunnel, which was accomplished in record time, and which contained some interesting engineering features.

The preliminary surveys for the tunnel line were made in 1902, and the first location was under Irving Avenue, Scranton, one block nearer the center of the city than the existing line, the object being to take advantage of the extensive old mine workings, known as the

* Presented at the meeting of May 16th, 1906.

† That portion of the paper describing the tunnel generally has been prepared by Mr. Francis, Civil Engineer for Westinghouse, Church, Kerr & Company, and that portion describing the contractor's plant and operations has been prepared by Mr. Dennis, Vice-President of the Rinehart & Dennis Company, General Contractors.

Dunmore No. 3 coal vein, cropping out just beyond the company's power plant. It was hoped that this might prove an easy, quick and inexpensive means of tunneling, but surveys in the mines demonstrated that the workings were not favorable to the accomplishment of this plan. The drift was above and below the tunnel grade and crossed the profile twice, introducing puzzling questions of support.

It was finally determined to locate a line which would avoid these old mine workings and clear the limits of the coal yet to be mined. The new line was located to the south of Irving Avenue, under private land contiguous to Crown Avenue.

The contract for a single-track tunnel was executed on June 1st, 1904, with the Rinehart and Dennis Company, of Washington, D. C. Time was the essence of this contract, which provided for the completion of the work, to a stage admitting of track laying throughout, in a period of 16 months, under a penalty and bonus clause providing for the payment of $200 per day.

Work was commenced on July 5th, 1904, after a brief delay on account of right-of-way matters. The first round of holes for the heading was fired on August 12th, 1904, at the north portal, about one month having been consumed in excavating the approach cut. The excavation was completed and the tunnel ready for laying track on July 18th, 1905, the contractors earning a bonus of $19 000.

From a knowledge of the geology in the vicinity, and a determination of the nature of the rock through which the tunnel would be driven, it was evident that some portions would require lining; but, at the time the work was let, it was not known how much.

Typical sections were prepared showing the method of treatment of expected conditions.

After the work had progressed for a period of six months it was quite evident that nearly all the tunnel was in rock of inferior quality, and would require lining of some character for nearly its entire length. The rock had a tendency to slab off or drop in small pieces rather than settle down on the supports in large masses.

To effect a saving in the initial cost of construction, it was determined to introduce some form of permanent timber lining which would take the place of the more expensive masonry section. Instead of placing the timber outside of a future masonry section, it was put in the position of the masonry section, thus effecting a con-

PLATE XIV.
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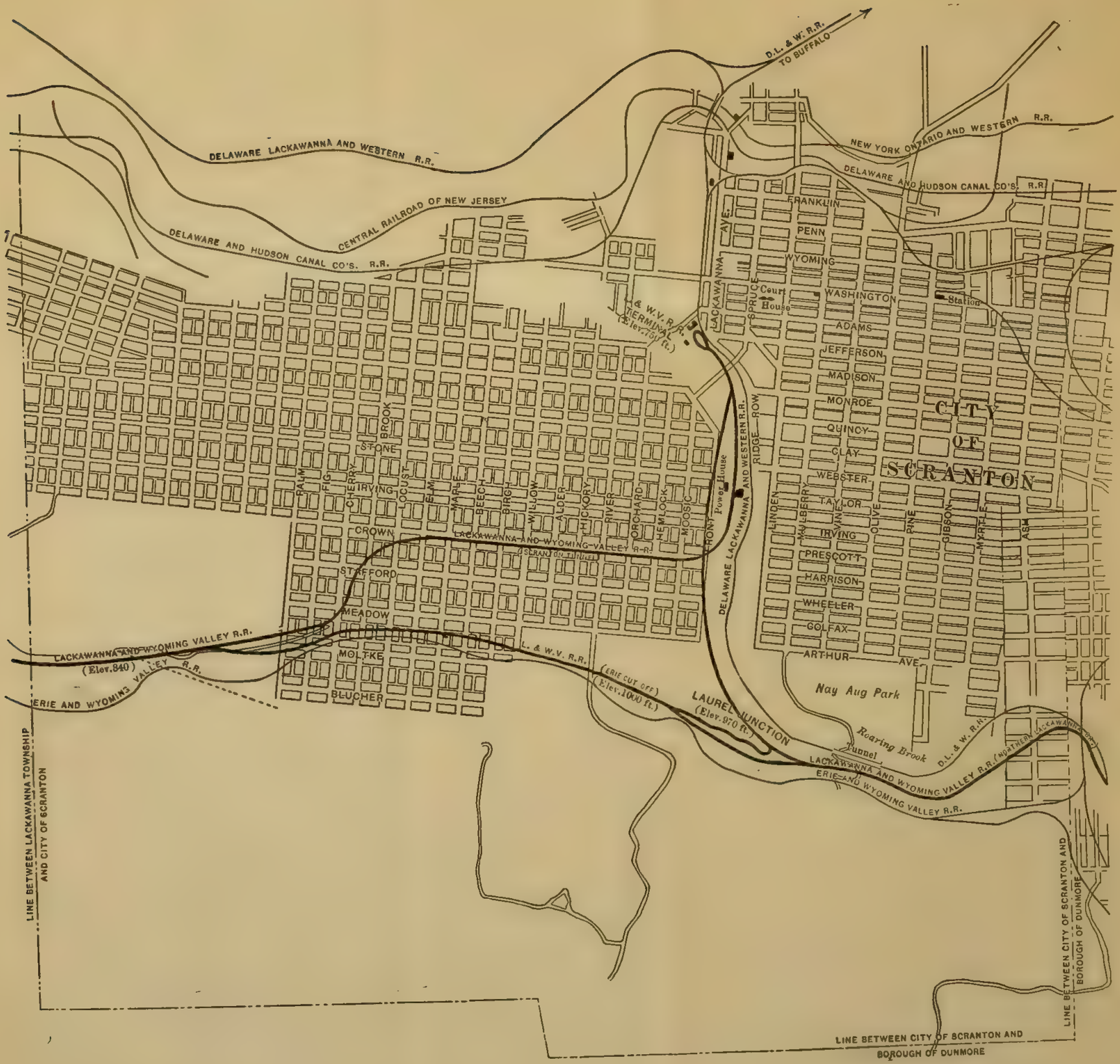

siderable saving, both in excavation and masonry. At such future time as it is desired to reline the tunnel, short stretches of the timber may be removed and replaced with timber or masonry.

The dimensions of the tunnel are as follows:

Total length, face to face of portal masonry.	4 747	ft.
Total length excavated as tunnel.....	4 640	"
Portal masonry extended at north end....	16	"
*Portal masonry extended at south end....	91	"
Width, in clear, inside.....	17	"
Height above top of rail.....	20	"
Depth below top of rail.....	2	"
Arch, semi-circular, radius.....	8.5	"
Shaft No. 1, 10 by 20 ft., depth.....	104	"
Shaft No. 2, 10 by 20 ft., depth.....	180	"

Surveying.—The alignment of the Scranton Tunnel presented some of the difficulties usually attendant on work of this character; the principal ones were:

1.—The center line of the tunnel at the surface passed through a great many buildings;

2.—There was no higher ground in the vicinity on the axis of the tunnel.

3.—The surface was quite uneven, and five permanent points had to be established on the prime base line on Crown Avenue;

4.—Owing to the configuration of the surface at the portals, curves had to be extended into each end of the tunnel;

5.—The shafts were located to one side of the center line of the tunnel, and came partly without and partly within the tunnel prism.

A prime base was established along Crown Avenue, 45 ft. from, and parallel with, the projected tunnel line. This line was run and re-run, reversing the transit telescope, differences were eliminated and adjusted, and permanent points established. Then, as a check on the work, a line was run over the work several times, using no back-sights.

After a base line with no apparent error had been established in Crown Avenue, tangents were brought ahead from the existing tracks at each end to an intersection with this base line; and, at

*Extended to carry the roadway over the tunnel.

the north end, a base 80 ft. long was obtained for measuring angles. Angles were turned a sufficient number of times to insure their correct valuation, and curves were run in and checked by direct measurements from tangents and by complete triangles, in which all distances and all angles were measured. These triangles were remeasured and adjusted until no appreciable error remained. By this method a permanent point was established on the center line of the curve at a convenient place near each portal.

From the points established along Crown Avenue, other points convenient to the shafts were put in place, and from these and at right angles thereto, monuments were established, about 51 ft. away, to take the line down the shafts. As the center line of the tunnel was 45 ft. distant, another offset was necessary at the foot of the shafts. It was complicated still further at these points, in the early stages of the work, by the presence of pillars or shoulders of rock left on either side at the bottom of the shafts for the protection of the hoisting apparatus, and two more offsets had to be made to get to the center line. After the removal of this rock the line was taken directly from the wires in the shafts by producing the lines about 200 ft. either way and repeating the operations until there was no appreciable error.

The method of taking the line into the shafts, as shown in Fig. 1, was simple, but great care was taken to make the lines accurate. An observer with a transit, with a fixed fore-sight, was always left on the surface of the ground. Plumb-bobs, weighing about 30 lb., were suspended with copper wires from points about 2 ft. above the curb, or top, of the lining of the shaft.

The wires were hung over a notched bolt, running through a U-shaped piece of iron, with a thumb-screw and spring adjustment. The plumb-bobs were steadied in casks of water or oil at the bottom of the shafts, and the casks had covers so that dripping water would not disturb the surface of the liquid. Even with this precaution, there was always some tremor in the wires, owing to the fact that they were frequently struck by falling water.

The lines met in the headings between the south portal and Shaft No. 1 with a variation of 0.16 ft.; between Shafts Nos. 1 and 2 with a variation of 0.02 ft.; and between Shaft No. 2 and the north portal with a variation of 0.23 ft. In no case was the center line, as run from the shafts, more than $1\frac{7}{8}$ in. from the actual center line.

FIG. 1.—NORTH APPROACH, SCRANTON TUNNEL. TUNNEL LINE ON LEFT. TEMPORARY LINE ON RIGHT.

FIG. 2.—SOUTH APPROACH, SCRANTON TUNNEL.

The grades met between the south portal and Shaft No. 1 with a variation of 0.02 ft.; between Shafts Nos. 1 and 2 with a variation of 0.07 ft.; and between Shaft No. 2 and the north portal with a variation of 0.05 ft.

Alignment and Grade.—At the south end of the tunnel a 4° 14′ curve, with a spiral 218.75 ft. long, was run in to connect the center line of the existing track with the center line of the tunnel. All the spiral and 270.34 ft. of regular 4° 14′ curve was inside the tunnel portal, or a total of 489.09 ft. of curve, was run in at this end.

At the north end, owing to a very narrow valley, a 10° curve, with spirals 400 ft. long, at each end, had to be introduced. One spiral, together with 186.21 ft. of regular 10° curve, or a total of 586.21 ft. of curve, was inside the tunnel portal.

At this end, as at the south portal, lines were given as the work progressed, so that the excavation for the heading was taken out to the full tunnel section.

The tunnel, generally, has a grade of 1%, but, for a short distance at the north end, it is 0.8 per cent.

Excavation.—The excavation was carried on at six different points of attack, two shafts being sunk, about 1 500 ft. apart, from which headings were worked in both directions, as well as at the north and south portals. Fig. 2 shows that 56%, or more than one-half, of the tunnel excavation was removed through these shafts, the work being expedited thereby fully a year in time.

The method of tunneling was in accordance with the usual American practice. The top heading was taken out to the full size of the section, 9 by 21 ft., and the length carried on in advance of the bench varied from 50 to 800 ft. The bench was afterward split in two lifts.

The quantity of excavation per linear foot of tunnel was as follows:

	Rock Section. Cu. Yd.	Masonry Section. Cu. Yd.	Timber Section. Cu. Yd.
Headings	4.71	7.04	8.38
Benches	9.00	11.00	12.00
Totals	13.71	18.04	20.38
Falling beyond payment line	1.00	1.13	1.18

Disposal of Spoil.—The portal excavation at the north end was dumped along the bank of Roaring Brook. At the south end the greater portion was loaded on cars and used as rip-rap at a point on the line of the road nearly 16 miles away.

The excavation from Shaft No. 1 was dumped on low land near by, purchased for the purpose; and that from Shaft No. 2 was hauled by surface incline to a dump about 1 200 ft. away and from 50 to 60 ft. higher.

The work was carried on by night and day in two 10-hr. shifts.

Lining.—A small quantity of temporary lining, outside of the masonry section, was required at each portal and at Shaft No. 1, at which points there was danger of immediate and frequent falls from the roof. At other places these falls were less frequent, but the roof would slab off to such an extent as to make it necessary to line, either with timber or masonry, before beginning the operation of regular train service.

Concrete masonry lining was adopted for those portions of the tunnel near the portals and near the shafts, where the rock was of the poorest quality. Lining of this kind, being the most expensive, was restricted as much as possible, and permanent timber lining substituted.

The permanent timber lining is of the usual pattern of voussoir ribs, 5 ft. on centers, with a 4-in. lagging. The yellow pine timber used for this purpose should last about 12 years before requiring renewal, and at the end of that time it can be renewed or replaced with masonry.

At three different points in the tunnel there were a few hundred feet where the rock was found sufficiently good to require no lining. There has been one fall, of moderate size, however, since the opening of the tunnel for regular traffic. These portions of the roof are being carefully watched, and if there is much uncertainty as to falls it will have to be lined.

The following are the lengths of the different sections:

Plain rock section	1 305 ft.
Timber-lined section	2 717 "
Masonry section	725 "
Total	4 747 "

PLATE XVI.
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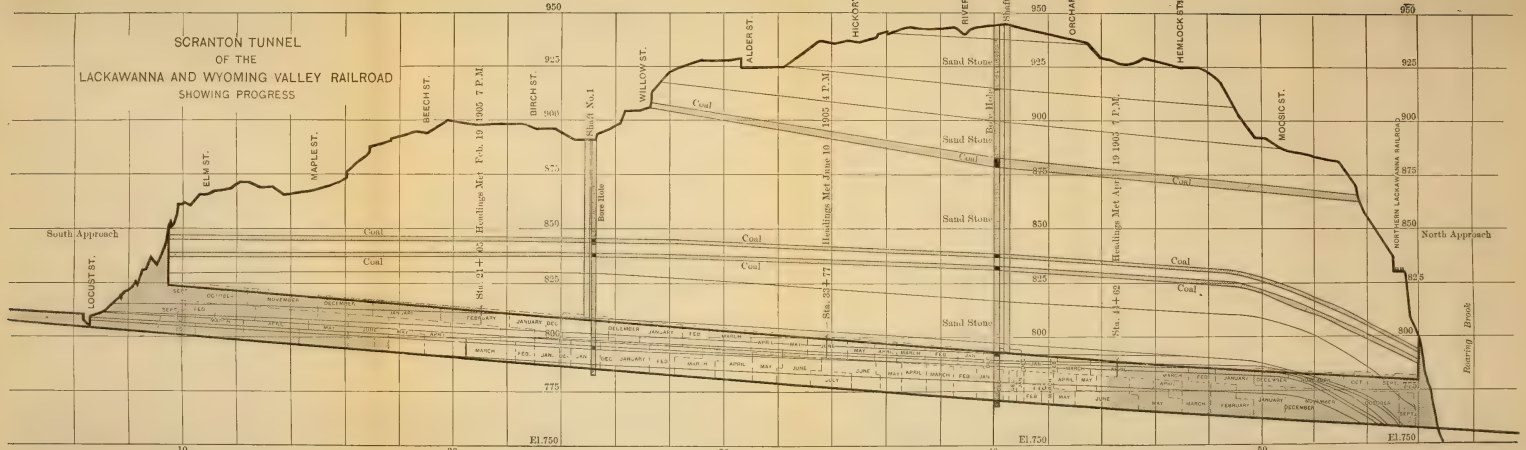


METHOD OF TRANSFERRING THE CENTER LINE DOWN A SHAFT.

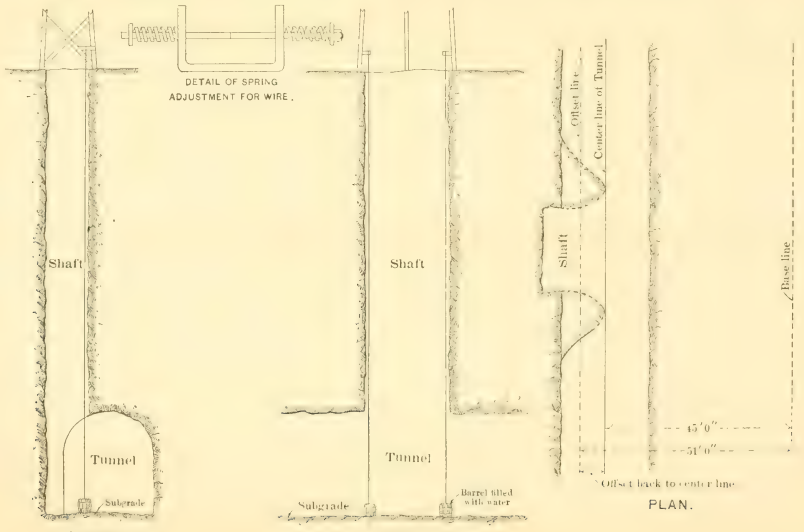


FIG. 1.

The proportions of concrete mixture for the two classes specified were as follows:

Class "A" concrete, in the arches of the roof or in the side walls where the thickness does not exceed 26 in., 1 part cement, 2 parts sand and 4 parts broken stone.

Class "B" concrete, in the side walls or tunnel arches, where the backing is rock in place, 1 part cement, 3 parts sand and 5 parts broken stone.

Table 1 gives the cement tests made.

TABLE 1.—Average Tensile Strength of Briquettes Tested.
Lehigh Portland Cement.

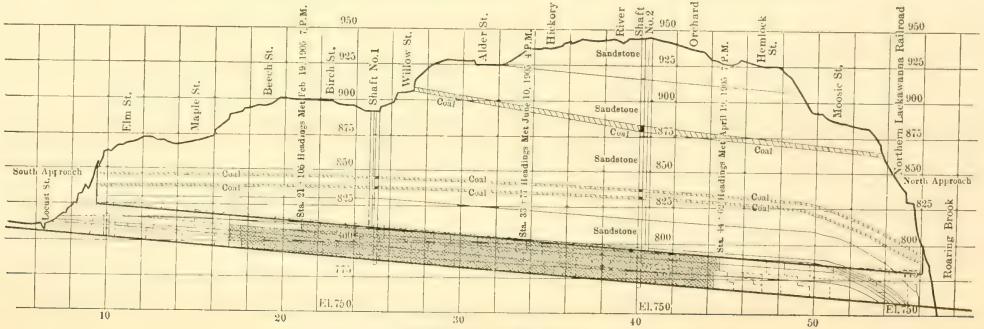
Consignment.		Briquettes made.	Fineness.		Boiling Test.	Mixture.				
						Neat, 20% Water.			1 Cement, 3 Sand, 10% Water.	
Car No.	No. of Barrels.		100 mesh.	200 mesh.		1 day.	7 days.	28 days.	7 days.	28 days.
L. V. 68 119	150	6	.....	.....	.....	.....	.....	.....	303	400
L. V. 5 379	150	18	89	72	N. G.	293	709	857	.....	.....
L. V. 71 827	150	30	87	69	O. K.	211	687	795	267	418
L. V. 64 005	150	30	90	75	O. K.	269	739	800	337	487
L. V. 70 298	150	30	90	73	O. K.	168	765	842	246	393
L. V. 64 198	150	30	90	73	O. K.	271	724	851	250	337
L. V. 61 602	150	30	90	72	.....	264	687	796	234	330
D. L. & W. 33 082	150	30	90	72	O. K.	301	742	854	300	434
D. L. & W. 26 354	150	30	90	70	O. K.	383	849	937	244	373

Note.—With the exception of the first item, the number of briquettes tested in each instance was six, and the figures given herein are the averages of the various tests.

Shafts.—There are two shafts (10 by 20 ft. neat size), which were located at available points, and divided the tunnel into three nearly equal parts. Shaft No. 1 has a depth of 104 ft. and Shaft No. 2 a depth of 180 ft.

These shafts were placed to one side of the center line of the tunnel, the object being to provide a place where the shaft leads and rigging would be out of the way and be less likely to be injured by blasting, thus obviating the resultant delays in the early prosecution of the work at these points. This location of the shafts was also a safeguard against possible accidents caused by dropping tools, etc., from above, or spoil from the cars while being hoisted. As a further protection to this hoisting apparatus against blasting,

SCRANTON TUNNEL: PROPORTION OF MATERIAL TAKEN OUT AT SHAFTS AND PORTALS.



Shaft No.1 . . . = 29 265 cu. yd.
" " 2 . . . . = 20 090 " "

Total by shafts = 49 355 " "
" " portals = 38 105 " "
" Excavation = 87 460 " "

56% taken out through shafts.

FIG. 2.

two shoulders of rock were left on either side of the shafts, and these were not removed until the tunnel excavation had advanced to points where the shaft rigging was safe from the effects of blasting.

This arrangement of the shafts was specially requested by the contractor, as it would enable him to lay the tracks in the tunnel in such a manner that the cars loaded with spoil, as they arrived at the mouth of the shafts, could, if found necessary, be shifted into either of the cages and raised to the surface without loss of time, thus securing the maximum service from this part of the equipment. In the actual prosecution of the work, however, this was not found necessary.

Each shaft was divided into two compartments by center cross-bracing, and these compartments formed the well for the cage or elevator, which was 8 ft. 7 in. square. The cages were built up of steel shapes, strongly riveted together, with a top cover of plate iron, the only wood about them being the floors. Channel guides were provided for the entire height of the cages, about 9 ft., and these engaged 5 by 8-in., dressed, hard-pine leads firmly attached to the sides of the shafts.

From each cage a wire hoisting cable, $\frac{7}{8}$ in. in diameter, was attached to the engine, both cables winding on the same drum, the load on the engine being lightened to the extent of the empty cage descending, which acted as a counterweight. As a safety appliance, the cages were directly cross-connected with a $\frac{3}{4}$-in. cable operating over separate sheaves, which acted as a direct counterweight. The hoisting engines were horizontal, two-cylinder, single-drum, steam-driven, and of 40 h. p.

The head-house, or hoisting frame, consisted of three framed bents, of 12 by 12-in. and 12 by 14-in. hard-pine timber, well tied together at top and bottom. Between this hoisting frame and the engine a guide with two sheaves was set up to keep the cable at the proper angle to feed to the drum.

One man was always stationed at the top and another at the bottom of the shafts, and no movement of the cages was permitted until they had interchanged signals, and the man at the top of the shaft had given the signal to the engineer. No accident resulted from the use of the shafts and cages.

The two shafts have been left for ventilating purposes; the rock

PLATE XVII.
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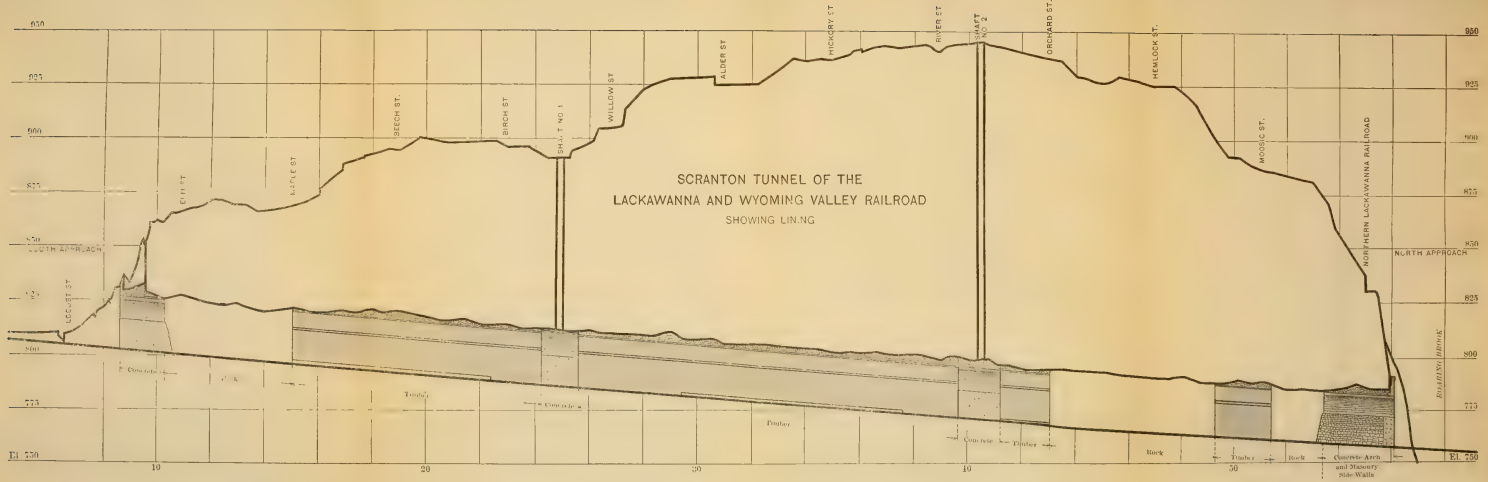

sides, being fairly sound, will not cave in to any extent, even if the timber lining, which has been left in place, decays.

The openings have been covered with crib houses, about 10 ft. high, and built of 8 by 8-in. second-hand timber, laid horizontally, and solidly drift-bolted. A flat roof-grating, of the same material, completes the structure, and provides a covering of sufficient strength to prevent anyone, especially children, from tearing off the boarding and getting into trouble by falling into the shaft. This method of treatment was much less expensive than building masonry structures.

General Progress.—The general progress is shown by the following:

Contract signed	June	1st, 1904.
Work started at Shaft No. 1	July	25th, 1904.
" " " " " 2	August	2d, 1904.
" " " South Portal	July	11th, 1904.
" " " North Portal	July	13th, 1904.
First round holes for heading fired at north end	August	12th, 1904.
First round holes for heading fired at south end	September	8th, 1904.
Excavation entirely completed	July	18th, 1905.
Meeting of headings between South Portal and Shaft No. 1	February	19th, 1905.
Meeting of headings between North Portal and Shaft No. 2	April	19th, 1905.
Meeting of headings between shafts	June	10th, 1905.

The progress is shown by the profile, Plate XVI.

Maximum Progress.—The maximum progress was as follows:

Maximum rate per month for all headings	881.5 ft.
" " " " " " benches	745.0 "
" " " week " " headings	237.0 "
" " " " " " benches	256.0 "
" " " month of any one heading	261.0 "
Maximum rate per week of any one bench	85.0 "
" " " month for shafts	111.0 "

Table 2 shows the progress in detail.

Lighting.—The tunnel is lighted by electricity, 16-c. p. incandescent lamps being installed at intervals of approximately 40 ft. throughout. The lamps are arranged in groups, with six 110-volt lamps per group, all lamps of each group being in series between the third rail and the track rail, and are located as high as possible on one side of the tunnel walls. Every sixth lamp is connected to the rail circuit with a No. 4 Brown and Sharp bare copper wire, which is securely fastened to the nearest tie and to the side wall of the tunnel in such a manner as to be entirely clear of moving equipment and safe from disturbance by the track men.

There is one feeder wire, of the same polarity as the third rail, running from end to end of the tunnel, to which each group of lights is tapped at every sixth lamp. The connection from the last lamp of each group is run directly to the track.

The positive feeder is connected to the third rail through a single-pole, 500-volt, 25-ampere, quick-break, knife-switch, and one 25-ampere, open fuse, mounted on a 50-ampere D. & W. fuse-block, all enclosed in a water-proof, wooden box, at the north portal, by which control is had of all the lamps in the tunnel from a single convenient point.

A 500-volt, 3-ampere, porcelain, open-fuse cut-out, of D. & W. make, is placed in each tap between the feeder and the first lamp. This affords protection to each individual circuit, so that an accident, which would cause a short circuit of any one of the independent groups, would not throw the entire installation out of commission.

The cost of the installation was about $8 per light.

Track.—The double track, approaching the tunnel at either end, is gauntleted through the tunnel, and two third rails are provided for electric contact, thus making six rails in a single-track tunnel.

The track is of 90-lb., Am. Soc. C. E. section, for the running rail, laid with tie-plates; and of 75-lb., Am. Soc. C. E. section, for the third rail, on standard ties and rock ballast, 1 ft. deep under the ties, all rails being properly bonded for the return circuit.

Signals.—The Union Switch and Signal Company's electric train-staff system of block signaling is in use, for the prevention of

PLATE XVII.
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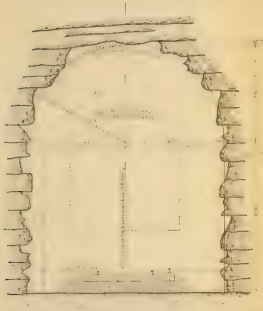
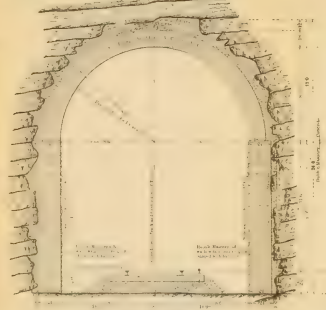
TUNNEL CONSTRUCTION

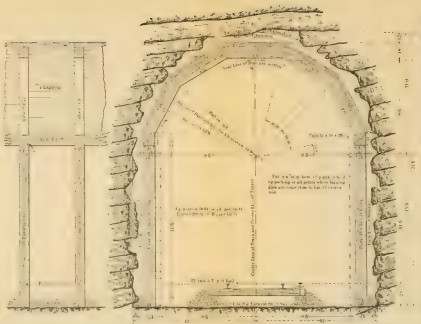
THE SCRANTON TUNNEL OF THE LACKAWANNA AND WYOMING VALLEY RAILROAD



SECTION WITHOUT LINING

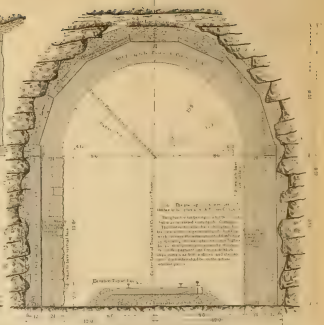


SECTION WITH LINING



SECTION WITH LINING

LONGITUDINAL ELEVATION OF TIMBERING



SECTION WITH LINING TIMBERED

TABLE 2.—Monthly Progress on Scranton Tunnel, in Linear Feet.

Date.	Headings.						Bench.						Totals.		Remarks.
Month of	South Portal.	Shaft No. 1.		Shaft No. 2.		North Portal.	South Portal.	Shaft No. 1.		Shaft No 2.		North Portal.	Heading.	Bench.	
		South.	North.	South.	North.			South.	North.	South.	North.				
July, 1904	Started July 11.					Started July 13.									9 + 56 to 55 + 96.
August						41							41		First round of holes fired Aug. 12.
September	88					135	24					69	223	93	
October	194	75	63	33	30	104	30	10	10			131	499	181	
November	261	13	17	11	26	216		46	40	10	10	196	544	302	
December	201	59	116	39	16½	130		47	56	29	31	110	561½	273	
January, 1905	254	140	156	97	92½	142		85	122	71	26	120	881½	424	
February	151	102	96	125	25	100	40	73	95	84	64	160	599	516	
	Met Fe	b. 19th													
March			158	103	126	124	175	164	149	105	45	107	511	745	
April			114	82	101	142	202	174	136	89	40	50	439	691	
					Met A	pr. 19th									
May			93	103			183	131	94	78	42	120	196	649	
June			70	75			100	27	124	105	87	143	145	587	
			Met Ju	ne 10th											
July								27	80	72				179	
	1 149	389	883	668	417	1 134	754	784	906	644	346	1 206	4 640	4 640	

collisions on this piece of single track, and thus trains are passed through with perfect safety.

Costs.—The following are the contract prices for the tunnel proper:

Shaft excavation	$7.00	per cu. yd.
Tunnel excavation	3.35	" "
Backfilling over timber and behind masonry	1.50	" "
Overhaul, 100 ft. in excess of 1 000 ft.	0.01	" "
Class "A" concrete in forms	9.00	" "
Class "B" concrete in forms	8.60	" "
Third-class masonry	6.50	" "
Long-leaf yellow pine	45.00	" M., B. M.

The average cost of the tunnel proper for excavation and lining, including the shafts, was $90 per linear foot.

Engineers and Contractors.—Westinghouse, Church, Kerr and Company, of New York, were the engineers for the work, and The Rinehart and Dennis Company, of Washington, D. C., were the contractors.

The execution of the work was under the personal supervision of Mr. P. B. Easterbrooks, Resident Engineer for Westinghouse, Church, Kerr and Company, and Mr. J. H. Rinehart, Second Vice-President and Secretary of The Rinehart and Dennis Company, to whom credit is due for its successful prosecution.

The purchase of the right of way, and the work of track laying, lighting and signaling were conducted by the operating department of the road, under the supervision of Mr. Charles F. Conn, Vice-President and General Manager.

Contractor's Plant and Operations.—Railroad delivery for coal, and a convenient water supply dictated the location of the air-compressor plant at the south end, and the compressor location fixed the situation of the camp, boarding houses and office.

The company's plans required two shafts, and the simultaneous excavation both ways from each shaft, and at the ends, thus making six points of simultaneous attack.

It was calculated that from 24 to 28 drills would be necessary,

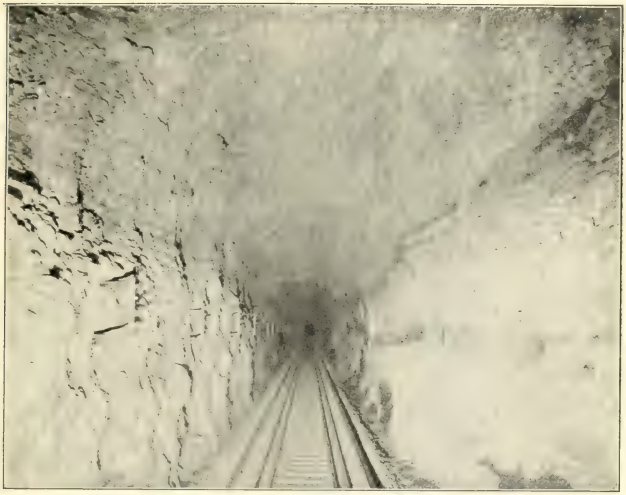


FIG. 1.—SCRANTON TUNNEL; PLAIN ROCK SECTION.

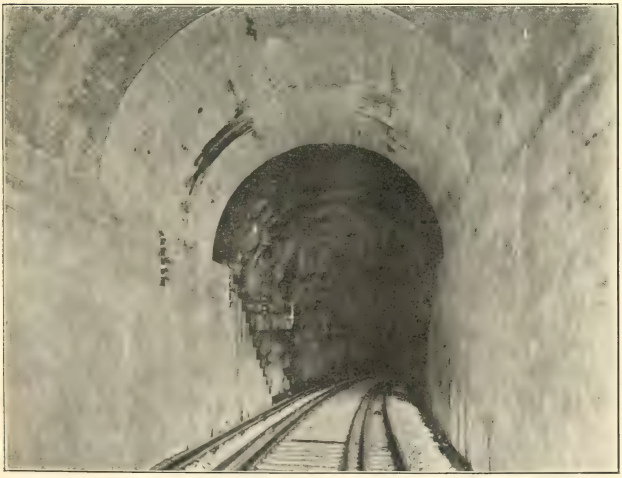


FIG. 2.—SCRANTON TUNNEL; JUNCTION OF PLAIN ROCK SECTION AND CONCRETE-LINED SECTION.

and the requisite air supply was approximated at, roughly, 100 ft. per min. for each drill, at 100 lb. per sq. in., and the boiler capacity at 20 h. p. to each drill. There was actually installed one 80-h. p. and three 150-h. p. boilers, all being return-tubular with brick arch, cast-iron fronts and iron stacks, making a total boiler capacity of 530 h. p.

The selection of the compressors was governed by the plant on hand, and comprised one Rand straight-line, 16 by 24-in., of about 600 ft. capacity, two Rand straight-line, 20 by 30-in., each of about 1 000 ft. capacity. At times all these compressors were run at about 20% more than their normal speed of 110 rev. per min., and, roughly, their combined capacity was increased to 3 000 ft. of air per minute when working at their maximum. The three 150-h. p. boilers provided an ample supply of steam, without the assistance of the 80-h. p. boiler, and the results indicate that the preliminary allowance was high, and that about 15-h. p. boiler capacity in this instance was sufficient to compress 100 ft. of air per minute to the gauge pressure of 100 lb.

Contributing to this economical boiler capacity was the use of anthracite coal under forced draft from a steam jet, making practically perfect combustion, and a more than usually tight distributing pipe line.

Whenever a pipe line is to be determined, there is always a balance to be drawn between small size, large friction, and small first cost, and the alternate. Without pretending to any accurate determination of these features, a practical balance for this particular work was to run a main 6-in., sleeve-connected, wrought-iron pipe from the compressor, past the first shaft, and up to the second. This pipe was reduced to 4 in. between the latter and a receiver at the north portal. A 3-in. pipe carried the line to the bottom of each shaft, and the further extension toward the several faces of attack was by 2-in. pipes resting on the bottom. Temporary connection between the end of each pipe and the drills was made by 50 ft. of 2-in. rubber hose, distributed to the several drills by 1-in. rubber hose. The main 6-in. pipe line was laid uncovered in the streets, over and parallel to the tunnel, and was laid during hot weather. On account of its position in the street, the pipe, while slightly sinuous in detail, was very nearly straight in its

general direction. Expansion bends in the pipe were impracticable on account of its location, and as expansion joints are costly and unreliable, the pipe was laid without any appliance to take up the change of length caused by variation in temperature. The line thus laid gave no trouble in passing through the changes of temperature from summer to winter, and from winter to summer.

The general method of tunneling was to carry the bench and top heading together, with the heading from 50 to 75 ft. ahead. A traveling framework, of half the tunnel width, and with the top at a lower elevation than the top of the bench, was mounted on wheels, which ran on temporary rails. The frame admitted the passage of two cars—one to be run up to the bench at its side, and the other underneath. Either car could be loaded by chutes from the platform, and, at the same time, by shoveling the bench excavation. The connection between the traveling platform and the unexcavated portion of the bench was formed with 4-in. lagging built into a plank 24 in. wide, thus making a wheel-barrow run-way between the face of the bench and the movable platform. The heading spoil was loaded into wheel-barrows, wheeled over the plank and dumped through the chute on the traveling frame, the whole operation being performed without interfering with the loading of the bench spoil.

When it became necessary to blast, the planks were simply loaded on the frame and the latter moved back on its wheels to a safe distance.

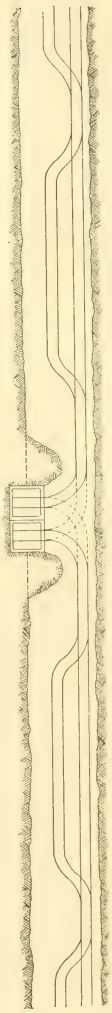


FIG. 3.

CONTRACTORS LAYOUT OF TRACKS AT SHAFTS FOR HANDLING SPOIL.

FIG. 1.—SCRANTON TUNNEL; MASONRY-LINED SECTION, WITH RUBBLE SIDE WALLS AND CONCRETE ARCH.

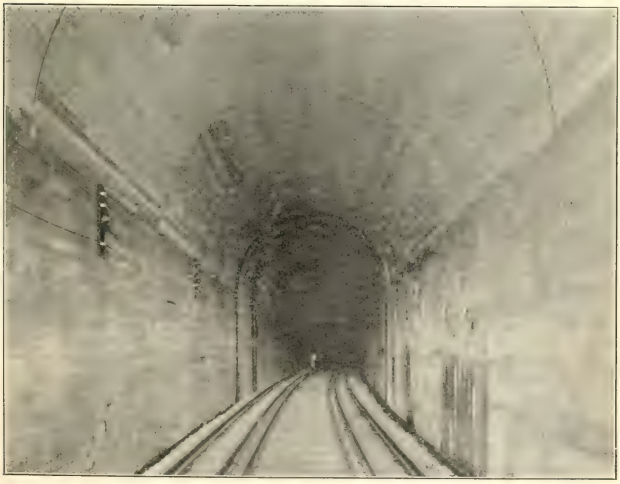


FIG. 2.—SCRANTON TUNNEL; MASONRY-LINED SECTION, CONCRETE THROUGHOUT.

The foregoing method seems to be so simple and effective as to be scarcely worth description. Other ways of handling bench and heading material at the same time are advocated; nevertheless, the method outlined seems to the writers to be the simplest and cheapest.

A peculiarity of the bench rock, in part of this tunnel, modified considerably, not only the foregoing procedure, but also affected the timbering. That peculiarity was a consequence of the combined hardness and tenacity of the bench material. To break it out required so much explosive that the rock was blown lengthwise of the tunnel with such force as to wreck any permanent timbering erected within 200 ft. In addition, block-holing the spoil was always necessary after the first blasting. The interruption to the work of loading the heading spoil, caused by moving the traveling frame out of danger, was so serious as to render it impracticable to continue the simultaneous excavation of the heading and bench. The heading material, while requiring timber for permanent support, could be left temporarily unsupported; the support being required, not to hold up an overhead mass, but to prevent and support exfoliation, slabbing and weathering of the material. On account of these features, the heading was worked for a reasonable distance ahead, and the force then dropped back and split the bench in two lifts.

The full section of the tunnel was carried without timber support, to the extent of as much as 300 ft. The timbering was then erected from the bottom, and its full section was completed and packed from the floor of the tunnel.

Drilling and Shooting.—The typical plan of drilling the heading was, as shown in Fig. 4, with the eighteen holes fired in the order thereon indicated. From two to six additional holes were often found necessary. The longest holes were the cut holes fired first, which were from 8 to 9 ft. long. The widening holes were from 6 to 8 ft. long. The round was counted to make an advance of from 4 to 6 ft.

With two $3\frac{1}{4}$-in. machines, the time required to drill the holes was generally 7 hr. The total round drilled averaged about 140 lin. ft. The completion of the loading, wiring, firing by battery in series, reconnecting for the successive blasts, and the delay necessary between them for the explosive fumes to be blown out by com-

pressed air allowed to escape for this purpose, consumed varying times, from 30 min. to 2 hr., or an average of, say, 50 or 60 min.

The delay from shooting was greatest in the portion of the tunnel excavated from the shafts. In this portion the fumes seemed to hang and accumulate, not only on the firing side, but also on the other side of the shaft, producing delays in both places.

The explosive was 40 to 50% dynamite, mainly the latter, and the cost per cubic yard, for explosives, caps, wires, etc., for all the excavation, was equivalent to the cost of $3\frac{1}{2}$ lb. of 40% dynamite.

Cars and Rail.—The type and capacity of the dump cars used in tunnel excavation is a practical question of considerable importance. General conditions require a 3-ft. gauge. The cars have to be handled as single units up to the portals or shafts, and beyond. No method seems to be economical, except by hauling single cars by mules. The car, therefore, should be nearly equivalent to the hauling capacity of a mule, and must be open at the end so that in loading the least lift is required to reach its floor—the dumping will require change or reversal of position. The car, therefore, must be rotary. A car of the following description answers well in practice:

A rigid frame, with four 15-in. wheels, $3\frac{1}{2}$-in. tread, $2\frac{1}{2}$-in. axles; inside bearings, iron rotating spider; bed about 5 ft. 6 in. at back, 5 ft. 8 in. at front, 6 ft. long, with 18-in. sides, all inside measurements. Such a car transports about 1 cu. yd., solid, and weighs, empty, about 2 000 lb. The car is fitted with a removable tail board, like a cart.

A 20-lb. rail, with abundant cross-ties, can be used; but a 30-lb. rail is the more economical, in the end. Rails, in order to be handled down the shaft, should be in lengths of not more than 20 ft. The detail of moving ahead, in order to get the car up to the bench face, is arranged by placing, inside of the fixed rail, loose rails laid on their sides, with their heads against the web of the fixed rail. The car wheel flanges roll on the web of the loose rails, and the latter are slipped ahead at intervals until the excavation permits another length of fixed rail to be laid.

Timbering.—It will be noticed, Plate XVIII, that there were plans for two methods of timbering. One, the contract plan, was for solid timbering, with provision for concrete arching against its inside perimeter; the other was for voussoir blocks erected inside of

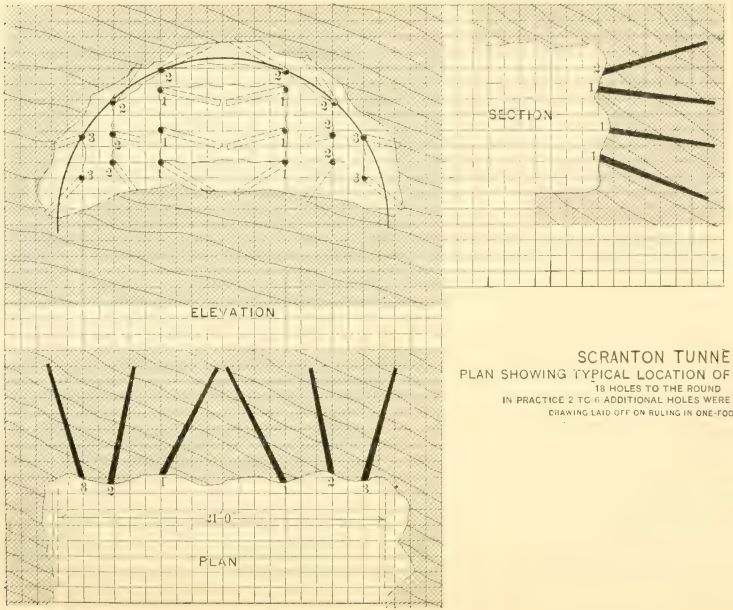

ELEVATION
SECTION
21-0
PLAN
SCRANTON TUNNEL
PLAN SHOWING TYPICAL LOCATION OF BLASTING HOLES
18 HOLES TO THE ROUND
IN PRACTICE 2 TO 6 ADDITIONAL HOLES WERE OFTEN NECESSARY
DRAWING LAID OFF ON RULING IN ONE-FOOT SQUARES

FIG. 4.

the masonry section and intended to be removed at some future time and replaced by concrete. Timber arching of 10 or 12-in. square arch blocks, with 3 or 4-in. lagging spanning the space between each pair of rims, is the standard of long experience. For economy and ease of erection and packing, and subsequent stability, it cannot be improved.

The contract form of timbering, here designated as "segment lagging," for want of a better name, in which the lagging and arch blocks are, so to speak, the same, is ideal in some respects. It uses more timber per foot of tunnel, but gives stronger support per foot, saves in single-track tunnel, roughly, 1 cu. yd. of excavation per foot, and, where the tunnel is to be lined with concrete, furnishes a back form for the concrete, closely concentric to the soffit, thereby saving either an excessive use of concrete to fill the space between the masonry arch and the timber, or avoiding the formation of an extrados for the concrete where the arch is held to a regular thickness. It also avoids the subsequent expense, uncertainty and delay of packing about 1 cu. yd. per lin. ft. between the concrete and the timber.

Pre-supposing a short length of tunnel provided with this timber in place, it would seem to be an easy matter to add to it, with the result of fitting the successive segments to perfect line and stable position. Trouble comes from the variation in the thickness and squareness of commercial timber and from the difficulty of getting a true radial joint. The effect of any inexactness of framing is to carry the bearing on part of the timber and leave other pieces loose. In all timbering the integrity of its form and stability in position when it is packed depends upon the perfect wedging between the voussoir blocks and the perimeter of the excavation, assuming that the wall-plate is immovable in its position. When the arch block and lagged timber are used, the blocks require to be wedged, or propped, to the roof only for every 4 or 5 ft. of the length of the tunnel, then the lagging simply has to be laid on, and the packing space is free to be filled behind and around the props. The operations are few enough to permit the propping to be done thoroughly, without expense and delay.

With the "segment lagging," a very much greater number of pieces require to be wedged up. In the case of the 8 by 8-in. seg-

FIG. 1.—SCRANTON TUNNEL; PERMANENT TIMBER-LINED SECTION.

FIG. 2.—SCRANTON TUNNEL; SOUTH PORTAL.

ment lagging of the plan, as against rims at 5-ft. centers, they are seven and one-half times as numerous per foot of tunnel. This wastes timber in props and wedging. The supports form an almost continuous line, converting the packing space into a number of separate pockets, lengthwise of the tunnel, in which thorough packing is difficult. On the other hand, in any form of timbering, the packing will settle away from the excavation, due to shrinkage of the timber, or shrinkage and readjustment of the packing material from the jar and concussion of blasting, or frequently from the settlement of the wall-plate itself. With the arch-block timbering, there being a free space above the timbering, the tendency of the packing is to settle and slide toward the haunches and open the inside joints of the haunch-blocks. With the segmental lagging, the settlement is confined to the separate longitudinal spaces between the lines of props, and does not become cumulative; therefore, there is less tendency toward deformation.

The experience of the writers has been that the "segment lagging" timbering costs very much more per thousand than the arch-block form. It should also be noted that the yard of excavation saved is not saved at the full price for tunnel excavation. All that is saved is the cost of loading and disposing of the spoil, and of a very small portion of explosive. The cost of all drilling, power and plant, and the general expense and profit, would remain the same per foot of tunnel.

If the company had desired to put in construction or permanent timbering, and to leave sufficient space to put in, at some future time, a masonry lining, without removing the original timber; and if it had had the option of putting in either the "segment lagging" form or the arch-block form of timbering, and could have paid the same unit prices for excavation, timber and packing in either case; there would have resulted a very material saving in cost per foot of tunnel by the use of "segment lagging," and, in the writers' opinion, a very much stronger timber arch would have been obtained during such time as reliance was placed upon it. With the Scranton prices and dimensions, the writers would estimate this saving, for equal masonry clearance, at about $2 per ft., if the same unit prices held.

For the reasons before stated, the contractor's unit prices, if he had the chance to consider the two forms in making proposals,

would have to be higher for excavation, timber and packing in the case of "segment lagging." The experience of the writers is that these higher unit prices would more than overcome the apparent saving. In other words, if offered the construction of a single-track tunnel at a given price per linear foot of tunnel, with either form of timbering at the contractor's option, the writers would prefer to use the arch-block form, although it is believed that the cost of the two forms would be nearly equal.

Practical erection difficulties in "segment lagging," in its requirements for exact radial joints, and in its multiplication of wedging, would lead to preference for the arch-block form as being more rapid and safe with rougher workmanship.

The foregoing comparison applies only to the method of timbering for average shale or rock tunnel, without consideration of its adaptability to masonry lining to be built afterward.

If the "segment lagging" timber is to be lined afterward with a concrete arch, the practical advantages of this form of timbering, in reference to future concrete, are very great. The advantages are mainly in reference to packing with solid concrete the whole space between the soffit of the concrete arch and the perimeter of the timbering with the minimum quantity of material, and, at the same time, holding very closely to some pre-determined thickness of arch. If the arch-block timber be in place, and the same thickness of arch concrete is required, there is the 12-in. space between the lagging and the extrados of the concrete to be filled in addition. If filled with concrete, there is required an extra quantity of from $\frac{3}{4}$ to 1 cu. yd. per ft.; if with dry packing, the same quantity, and with a support built up of concrete, packing, timber and again packing. By the "segment-lagging" method, there is the original dry packing above the timber, and everything below the timber is solid, making a better job with a saving of the intermediate packing.

Lighting during the Tunnel Excavation.—The writers have generally used gasoline (1-gal. tanks with open burners) for lighting tunnel work on the ground. Its portability, the opportunity of obtaining illumination at the right place, and its general flexibility overcome its high cost.

Gasoline, of course, is dangerous; the renewal of the lamp is a

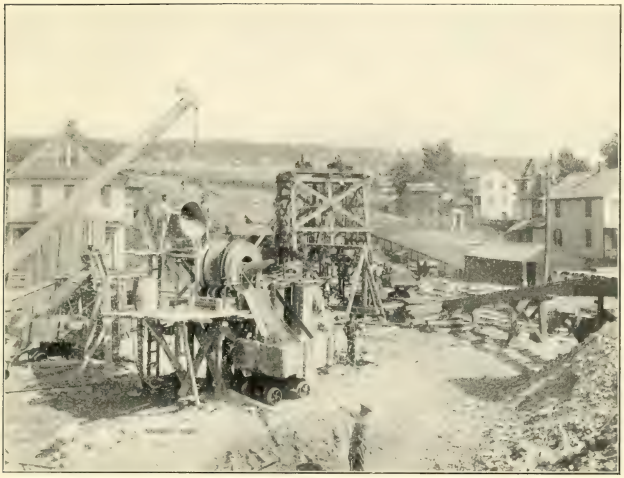


FIG. 1.—SCRANTON TUNNEL; HOISTING RIG, SHAFT NO. 1.

FIG. 2.—SCRANTON TUNNEL; SHAFT-HOUSING.

considerable expense, and occasional accidents are bound to occur. The heat, also, is very objectionable.

The accounts show that the cost of lighting, night and day, inside and outside of the tunnel, was more than $6 000, or, roughly, say, 6 cents per cu. yd. By months, it represents $500 per month for double-shift work, or, in another form, about $1.25 per ft. of tunnel.

The contractors returned to the use of gasoline after a previous experience with a tunnel of similar length but smaller section, driven with the use of the electric light. As the tunnels were of greatly different sections, and varied almost totally in material and method, the comparison is not at all fair, but electric light on that work cost about 11 cents per cu. yd. With the electric light there is a perpetual nuisance in replacing wires and broken globes, and it is troublesome to concentrate the lighting where it is needed most—right up in the heading at the front.

It is believed that, on an average, there will be found no material cost advantage in one method over the other, and the advisability of the method to be adopted will have to be settled by its facility in use. The writers have thought, heretofore, that the verdict was in favor of gasoline, but now believe that the electric difficulties can be overcome in part, so that, at least for long tunnels, it will be the better method.

DISCUSSION.

Mr. Lavis

F. Lavis, Assoc. M. Am. Soc. C. E.—It seems to be the general impression that tunnels are seldom completed within the contract time; this paper, therefore, is interesting, in that it cites an exception, and also gives many practical and valuable details of the progress and methods of work. The incentive of the bonus evidently had the desired effect of hastening the completion, and that by as much as 95 days, or nearly one-fifth of the allotted time.

In view of the general difficulty of enforcing penalties for the non-completion of contracts within a specified time—unless actual damage can be proved—the speaker would like to ask Mr. Francis if he thinks the penalty clause, of a bonus and penalty provision, could be enforced in the courts, if such action were necessary, without reference to any actual damage sustained. It is not stated in the paper, but it would appear, from the amount of the bonus, that an allowance was made covering the delay due to the final settlement of right-of-way matters, and the contract time did not start until after these had been adjusted, some 20 days after the contract had been signed. As this is a matter of considerable interest, it would be well to have it made quite clear.

In regard to the progress, it would seem at first thought that a record had been made in building a tunnel nearly a mile long in less than a year, and, in comparison with the majority of tunnels built in the United States, it is a record. Tunnels have been built in the United States, however, at a greater average rate of speed, and, of course, the long Alpine tunnels completely overshadow anything done elsewhere. For this reason it would be particularly interesting to know why this is so, particularly as the paper presents the case from both the engineer's and contractor's standpoints.

It is very easy to determine, theoretically, how many feet can be drilled by each drill per shift, how many drills can be worked at once in each heading and bench, and therefore what the progress ought to be. There should be no trouble in handling the muck, in any quantity, with steam shovels or other modern mechanical appliances, and this gives a record (on paper), far ahead of anything actually accomplished, at least in the United States. In Europe, apparently, theory is borne out in practice; in America, apparently, it is not—or, at least, thus far, has not been—and it would be interesting to hear, from the standpoint of the contractor, why this is so. It is hoped that Mr. Dennis may be able to throw some light on the subject.

In comparing the progress in any two or more tunnels, differences in cross-sectional area and in material penetrated must

be considered. Objection may be made to comparing the record of tunnels 1 or 2 miles long with the long Alpine tunnels; inasmuch as, in the latter, the large amount of work to be done justifies most elaborate preparations and plant, but the difficulties and length of transport to and from the working faces as well as the other enormous difficulties of bad ground, large quantities of almost boiling water, and high temperature throughout, more than justify the comparison on practically an even basis. The possibility, or otherwise, of sinking shafts, and thus obtaining several points of attack, must also be considered; the only fair basis of comparison, therefore, is the amount of work done, reduced to terms of one working face. Mr. Lavis.

In the Scranton Tunnel, on this basis, the actual working time on the tunnel proper, after the approaches had been completed, was about 11 months, which means an average progress, from each of the six working faces, of about 70 ft. per month. Comparing only the time worked on the headings—that is, the sum of the months worked in each heading—the average progress of the headings was about 103 ft. per month. The average progress on each bench, taken in the same way, was about 86 ft. per month.

In the construction of this tunnel no especial difficulties seem to have been encountered which would preclude its comparison with the following tunnels built in America; the sandstone and shale of the coal regions is not usually hard to drill, and the fact that 18 holes in the heading could be drilled by two drills in 7 hr. seems to show that it was particularly easy. If rapid progress was an object, especially when it was considered necessary to work the heading and the bench separately, there seems to be no reason for not using at least four drills in the heading, two on each column, as two columns had to be set up in any event, or a single column shifted from one side to the other. No material delay should have been caused by the timbering, as apparently there was no heavy ground, and the timbering could be erected at the contractor's convenience (within reasonable limits).

Comparing the progress on the Scranton Tunnel, therefore, with the progress made on other American tunnels, it is found that the average for the headings in the Hoosac Tunnel, was 133 ft. per month in the east heading, for the four years, 1869 to 1872; 120 ft. per month for the west heading, for the four years, 1870 to 1873; and, from the central shaft, 102 ft. per month for the two years, 1872 to 1873.

The Stampede Tunnel was built in 1886 in an extremely inaccessible region. It is 9 950 ft. long, with a section of about 15 cu. yd. per lin. ft., and was completed in 28 months; 6 of

Mr. Lavis. which were occupied in getting the plant on the ground. After the plant was erected, a length of 9 050 ft. was driven from only two working faces—no shafts being possible—in 22 months, or an average rate of 206 ft. per month from each working face. The whole length was timbered, and considerable trouble was caused by the swelling of the shale because of exposure to the air. This is believed to be one of the best records made in America.

The Cascade Tunnel was built in 1897-1900. It is 13 813 ft. long, and is a single-track railroad tunnel. It was through medium hard granite, very seamy and wet, necessitating timbering throughout, and was driven in 38 months, from only 2 headings, or at an average rate of 182 ft. per month from each working face.

The Croton Aqueduct Tunnel, built between 1885 and 1890, for the water supply of New York City, had a cross-sectional area of about 10½ cu. yd. per lin. ft. The average progress, for a period of 427 weeks, was 30 ft. per week, or about 130 ft. per month.

There are several recorded examples of progress of about 100 to 110 ft. per month in single-track railroad tunnels, but the general average is much lower, fully justifying Mr. Francis' claim for a record on the Scranton Tunnel; in fact, with few exceptions, the progress on the Hoosac Tunnel during the last few years of its construction has not been greatly improved upon during the thirty-five years which have elapsed since that time.

Turning now to European practice, and comparing the four long Alpine tunnels, a marked improvement is to be noted in each successive tunnel, culminating in the record for the Simplon of an average of practically a mile a year from each heading.

In the Mont Cenis Tunnel, built between 1860 and 1870, the speed was about 105 ft. per month from each heading; in the St. Gothard, built between 1872 and 1882, the speed was 225 ft. per month; in the Arlberg, built between 1880 and 1884, the speed was 350 ft. per month; and in the Simplon, commenced in 1898 and only recently finished, the speed was 440 ft. per month. All these are average rates from one working face for the whole length of the tunnel, rates as high as 700 ft. in a single month in one heading being recorded for the Simplon.

In a comparatively short double-track tunnel at Barrientos, Mexico, built in 1903, by native labor and with only a very small plant, 735 ft. were excavated in 92 days, or an average of 8 ft. per day. This time included holidays and time lost between Saturday noon and Monday noon of each week, when the men would not work. Of this tunnel 81% was driven from one end, so that the record is rather remarkable, being equal to about 200 ft. per month from each working face for a double-track section.

Mr. Lavis.

The extremely high rate of speed in the Alpine tunnels has been obtained where the Brandt type of rotary hydraulic drill, mounted on a carriage, has been used. Although these drills certainly show a high rate of efficiency, and have great capacity for rapid work, the speaker hardly believes that this explains the whole difference. He is rather inclined to think that the difficulty is principally with the labor question, and, to a certain extent, the indifference of the American contractor, as a rule, to the smaller refinements in keeping his plant and equipment up to a high state of efficiency, caring for the comfort of his men, and establishing an *esprit de corps* among them and a personal interest in the success of the work. On this point the speaker hopes Mr. Dennis will give some further information, from the standpoint of the contractor.

The variation in the progress from month to month is one of the interesting features brought out by almost any progress profile of tunnel work, and the present case is no exception. Part of this can probably be accounted for by delays incident to timbering and the change between sections which required no support and those where a lining was necessary, but, in the section between the two shafts—which is apparently all more or less the same—eliminating the first two months after the shafts were completed, so that a fair start could be made and the short section of masonry lining completed, it will be seen that in the headings the rate of progress varied from 82 to 156 ft. per month, and in the benches from 78 to 149 ft. The query naturally arises, if 150 ft. can be done in one month why not in every month? This, perhaps, can be asked with some reason, in this case, as in one month 261 ft. of heading were driven, and, in one case 202 ft. of bench. As a matter of fact, in actual practice, this uniform rate of speed, even where natural conditions are practically the same, does not seem to be attainable. The variation seems to be most often due to causes other than the natural difficulties encountered in construction. During some months apparently everything goes all right, a full force of men is obtained and kept at work, the plant works smoothly, and a record is made. During the next month, there may be days when the air supply is bad, the cages or hoisting apparatus get out of order, supplies do not arrive when expected, drillers are scarce, and things happen which are apparently beyond the control of the contractor, or anyone else, but they all tend to keep the amount of work done more or less below the maximum.

It has seemed to the speaker that, in order to attain a high rate of progress on this class of work, almost military discipline is necessary, and it is only by attaining a clockwork regularity in

Mr. Lavis. all the operations that anything like the record of the Simplon Tunnel can be obtained. It is needless here to point out the differences in operation between the method used there and the ordinary American method, as those interested can find many full accounts in current engineering literature; but it would seem that better control of the labor and discipline of the men was possible in Europe. This, also, is probably the reason for the excellent progress in the Barrientos Tunnel in Mexico, notwithstanding the generally though incorrectly supposed inferiority of the individual laborer. Men employed on tunnel work in America, especially drillers, seem to be difficult to get and hard to keep; the available force not only varies in numbers, but the personnel varies from day to day, and it is safe to predict that after each pay day there is apt to be an entirely new set of men in each heading, or none at all at times.

As noted previously, the speaker is inclined to think that some of the lack of success in attaining as high records in American as in European tunnels is, to a certain extent, due to the fact that American contractors, as a rule, consider it a useless refinement to attain any great amount of perfection in equipment or to keep it in a high state of efficiency, nor do they take much care for the comfort of the men, in the way of providing proper drying rooms, lockers, and accommodations for changing their clothing when entering and leaving the tunnel; or in providing adequate ventilation, not only to secure quick access to the work after blasting, but that the men may work at their highest efficiency, which they can hardly be expected to do in an atmosphere filled with dynamite fumes. The speaker has had occasion, recently, to compare organizations based on very different ideas in this regard, and is inclined to think that a certain amount of care for the health and comfort of the men, adequate ventilation of the work, and an equipment kept in first-class order pays for itself, in increased output and *esprit de corps*. If speed is the prime requisite, the extra expense necessary to get and keep a picked body of men is, of course, of no consideration; but, even from the ordinary standpoint, the decreased length of time during which the general expenses of a contractor, plant running, superintendence, etc., have to be met, balances to a large extent the money spent in hurrying the work to completion, to say nothing of the added prestige obtained by a contractor who does rapid work.

That the lack of ventilation is one of the incidental causes of delay is shown by the statement that some time was required to clear the tunnel after blasting, especially at the shaft, where blasting in one heading not only caused delay there, but also in the opposite heading. No considerable use seems to have been made yet of

the combination of compressed air and water jets for laying the dust and fumes of blasting, which it is reported has been used with considerable success in England and in the Simplon Tunnel. Mr. Lavis.

In the matter of handling the remaining section of the tunnel after the heading has been driven, the European method, in hard-rock tunnels, of driving a bottom center heading, then driving a top heading in both directions from an upraise, and thus getting several points of attack, while apparently more expensive, seems to achieve good results; but the speaker believes that by the use of steam shovels, or rather air shovels, the American system of driving a top center heading and then widening out and running a sub-bench and bench, can be worked up to a high rate of speed, provided a permanent force of good men, with efficient foremen, be kept constantly at work to provide the muck. To achieve the maximum results, the power plant must be designed to give ample power, with a good reserve in case of the break-down of any unit, cars, locomotives and tracks kept in a high state of efficiency, and an ample supply of drills and drill steel kept on hand, so that there will never be any delay from inefficient machines or lack of sharpened steel.

The speaker is unqualifiedly in favor of electric lights for use in tunnels, but has no data as to relative cost. He has had occasion, recently, to observe the working of the ordinary banjo gasoline lamps, as well as Kitson lights, in a tunnel in which electric lights were afterward installed, and believes that, owing to the better illumination, to say nothing of the freedom from smoke, smell and bad air, a much greater efficiency is obtained from the laborers. They are able to see what they are doing, the foremen, also, can see what the men are doing, and there is no necessity to grope around in the dark for tools, etc. There is no difficulty whatever in concentrating the light, at least in getting plenty where it is wanted, and the loss in broken globes, wires, etc., does not seem to have been excessive thus far.

It would be interesting to know if any observations were made in regard to the loss of pressure between the compressors and the various points of supply on the long air line used. The paper does not mention the use of re-heaters on this long line, and the speaker would like to ask whether or not there was any difficulty on account of this omission, especially during cold weather. Did the contractors consider the supposed increased efficiency due to their use insufficient to warrant the expense of installation and maintenance?

In regard to the difficulty of working the heading and bench at the same time, the necessity of using the scaffold car might have been obviated by keeping the heading and bench practically

Mr. Lavis. together. By blasting all eight cut holes together, instead of leaving the two top holes until the second round (as shown by Fig. 4), practically all the cut can be blown over the bench, as is nearly all the material from the first round of side holes. After the second side round is fired a small quantity of muck is left in the heading, and this can soon be thrown over the bench by a few men, and drilling can be started again almost immediately. This method is being worked satisfactorily in sandstone, but the speaker has some doubts as to its efficiency in tougher rock.

The description of the relative values of the two types of timbering is certainly most instructive, and presents the matter in what is believed to be altogether a new light, at any rate an unfamiliar one. Of course, had the ground been heavy, thus preventing the removal of the timber before putting in the masonry lining, the method with the timber occupying the position afterward to be filled by the masonry lining would not have been feasible, and in such a case the "segmental lagging" method would probably have been the better.

The engineers in charge of the alignment must be congratulated on the results obtained under the difficult circumstances surrounding their work, and the authors of the paper certainly deserve the thanks of the Society, and especially of those interested in tunnel work, for the practical nature of the paper and the interesting descriptions of details which are too often omitted.

Mr. Hewes. V. H. Hewes, M. Am. Soc. C. E.—In 1881 the speaker had occasion to sink a shaft for mining purposes. It was in three compartments, each 4 ft. 6 in. in the clear, and through rock carrying considerable water. It was sunk 300 ft., with two cross-cuts at the 300-ft. level, one of 300 ft. and the other of 200 ft. Air drills were used in sinking and cross-cutting.

Whenever shots were fired it was necessary for the men to wait in the hoisting works at least half an hour before they were able to enter the shaft again, the delay being caused by the powder smoke. A 6-in. square box, planed on the inside, was put in and extended from the roof of the hoisting works to the bottom of the shaft timbering, where the last section was made telescopic. A $\frac{3}{4}$-in. pipe was attached to the air receiver and carried into the box about 6 ft. above the floor of the hoisting works. It was then turned up for about 4 ft. and drawn down to a $\frac{1}{2}$-in. nozzle, an air pressure of 50 lb. being used. In sinking the shaft the timbering was kept from three to four sets (18 to 24 ft.) above the bottom to prevent it from being broken or displaced. The lower sets of wall-plates were held in place by suspension rods and wedges between the lagging and the rock at the sides. To prevent the rock from being thrown up on the timbering, from which it might fall upon the men while at

work, a system of 6 by 10-in. lagging was placed across the wall- Mr. Hewes.
plates on the lower set of timbers, with a plank across the end of the lagging, and on this were placed upright posts wedged against the wall-plate above. Two of the pieces of lagging at the end of the shaft were left loose, so that the telescopic section could be dropped below the timbering. The miners called the device a "smoke elevator."

When it was used for the first time a miner went down the center compartment to remove the two pieces of lagging and drop the telescopic section below the timbers, while the other men waited at the surface. After waiting some 5 or 6 minutes, and not receiving any signal, they went down the south compartment and discovered the man at work taking up the remaining lagging.

Afterward, when holes were fired, the men went down immediately, and there was no loss of time. On reaching the 300-ft. level, the "smoke elevator" was extended into the cross-cuts, where it worked perfectly, making a comparatively inexpensive method of removing the smoke and avoiding any loss of time.

W. F. Dennis, M. Am. Soc. C. E. (by letter).—The questions Mr. Dennis.
as to progress involve so very many factors that it is almost impossible to generalize. In the present instance, it would have been very easy to assume that the progress would be measured by the heading progress. If 40 ft. per week were allowed as a reasonable advance, and there were six attacks, multiplication would warrant the expectation of 240 ft. per week; or that the whole tunnel would be done in 20 weeks, barring the organization time, in getting to work, and the terminal time for finishing up the scraps.

A detailed examination of the profile would show, first, that the progress from the portals in no way controlled the general progress. That was limited by the progress from the two shafts. Preparatory to this work, the shafts have to be sunk. Next, the headings must be driven a reasonable distance on each side, then the shaft must be sunk to the bottom and the bench extended, and all this must be done with a small force and careful shooting. For a long time the firing operations of one side prevent or interfere with those of the other.

From the progress profile it will be seen that in both shafts it was January 1st before approximately regular excavation began on the floor of the tunnel between the shafts. From the writer's observation, the progress of tunnel work is almost always faster in the heading than the bench; so that the time problem, on January 1st, was to excavate 1 600 ft. of bench and a proportionate amount of heading, handle the material from the shafts, and at the same time extend from the same shafts similar construction toward the portals. This might have been limited by the hoisting and

Mr. Dennis. dumping facilities—as a matter of fact it was not, but the service was very nearly up to the full limit.

Referring to the former assumption of 40 ft. of heading per week, the calculation would be 80 ft. per week for the two, or roughly, the required time would be 20 weeks. Now, the rate of the heading progress was much in excess of this—40, 50 and as many as 60 ft. per week—but, on account of the slower progress on the benches, and for various other reasons, this progress was not maintained continuously, the heading work being at times entirely discontinued. The final clearing up of the bench was on July 18th, so that the theoretical time of 20 weeks became actually 28 weeks.

A further inspection of the progress profile will show that the progress was much less at Shaft No. 2 than at Shaft No. 1, the relative extensions being as 3 to 2. The forces and the equipment were the same in each, but the progress as a whole from one shaft was about 50% greater than from the other. A statement will be found in the paper regarding a very hard, glassy sandstone in the bench. Nearly all this material was adjacent to Shaft No. 2, which accounts for the relatively slow progress there.

As all other progress was subordinated to the shaft work, which fixed the real limit of time, another and fairer way of placing the time matter would be to say that the real time at issue was the building of 1 600 ft. of tunnel in about 28 weeks from two end shafts after they were down. Therefore, the full tunnel progress at the critical section was at the rate of 125 to 130 ft. per month. It was physically possible to have done this elsewhere, but if done it would not have helped.

Another way of stating the time progress would be to say that the real time work was equivalent to building a 1 600-ft. tunnel with approaches in about 35 weeks from the time the order was given to go ahead; that time including the construction of the camp, the installation of machinery, excavation of the approaches, and the completion of such a tunnel.

There is no record of the drop in air pressure at the end of the line, but it has been assumed that it was about 10 to 12 lb. No re-heaters were used. The writer has used them in the past, and the theoretical advantages claimed seem to be entirely warranted, yet he has never seen anybody in practical work persist in their use after once trying them.

The method of blasting heading and bench in the same series, so as to throw all of the muck on the floor, is economically impractical, in the writer's opinion. With the customary height of heading and tunnels, the bench has to be shot in two lifts, making, with the heading, three tiers of shots; or, to avoid this, the heading has to be carried at an uneconomical height, if only one lift be taken

on the bench. The writer's experience directs him to hold the heading as low as possible, for a variety of practical reasons, and this leaves the bench too high to be drilled in one round. Mr. Dennis.

The use of steam shovels in single-track tunnels is not economically advantageous in any class of material—although in certain materials it may increase the progress somewhat. Their use in double-track tunnels is another problem, and in any material, except in very hard rock, is economical. Where the rock is blocky, a rotary derrick is cheaper and does the work better.

The writer is familiar with the remarkable progress of the Alpine tunnels, but has not informed himself as to the cost, and details of the operations.

Every tunnel develops some special experience. As a class, the men are as resourceful, and the machinery men are as inventive and ingenious, as in other lines of effort. Thousands of tunnels have been built, and the American practice of top-heading ahead, with the bench in one or two lifts, has by evolution become so general that the probabilities are that it is economically the best for all fairly firm material. Such observation and experience as the writer has had convince him that this is so—certainly for America—and that the improvement in sight is in a constant study to improve the details of appliances.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

Paper No. 1023.

THE ECONOMICAL DESIGN OF REINFORCED CONCRETE FLOOR SYSTEMS FOR FIRE-RESISTING STRUCTURES.*

By John S. Sewell, M. Am. Soc. C. E.

With Discussion by Messrs. Wilbur J. Watson, Clarence W. Noble, I. Kreuger, Richard T. Dana, C. A. P. Turner, Ernst F. Jonson, Leonard C. Wason, E. P. Goodrich, Edwin Thacher, H. T. Forchhammer, Arthur W. French, Irving P. Church, B. R. Leffler, George Hill, F. P. Shearwood, Mansfield Merriman, A. H. Perkins, Langdon Pearse, C. B. Wing, William Cain and John S. Sewell.

The extensive use of reinforced concrete during the past few years has resulted in the evolution of many formulas for the design of the various members of a structure, but more especially those subjected to transverse stress. Many of these formulas are purely empirical, while others are based on a more or less rational theory. The latter, almost without exception, are closely assimilated to those applicable to beams of continuous and homogeneous structure. The empirical formulas are generally based on one or more manifestly incorrect assumptions, but are quite simple in form and application, and they contain constants which, in large measure, correct the inaccuracy due to the fundamental assumptions. Cases are not lacking, however, in which such formulas, in the hands of any but experienced designers, might lead to dangerous results. The more rational formulas are usually somewhat complicated in form; while

* Presented at the meeting of February 21st, 1906.

this complication is more apparent than real, these formulas are more or less forbidding at first sight, and they give to the uninitiated an impression of extreme accuracy, not at all justified by the nature of the materials and the form in which they are combined.

It seems to the writer that formulas combining the simplicity of the purely empirical ones—such as those used by the engineers of the Hennebique systems—with all the accuracy attainable by the more rational ones, are easily within reach. To present such formulas, with methods of applying them to designs of minimum cost, and to emphasize the value of certain little used features of design, from a fire-resisting, and therefore really economical point of view, are the main objects of this paper. Most of the opinions, suggestions, and conclusions which follow, were more or less clearly expressed in the paper submitted by the writer to the International Engineering Congress, at St. Louis, and in his part in the subsequent discussions on "Concrete and Concrete-Steel."* Subsequent study and experience having strengthened the writer's opinions along certain lines there indicated, they are here presented in fuller form, in the hope that they may bring forth a discussion which will at least assist in clearing up some obscure points.

In the design of rolled steel beams, quite accurate formulas based on the theories of elasticity and flexure are used, and the conditions fully justify it; but in the design of a plate girder, the lack of homogeneity, due to assembling with rivets, has led to a much simpler procedure—it is a question of simple shear in the web, and of the statical moment of the flange areas multiplied by an average unit stress. Unless the deflection is of importance, the elasticity of the material does not enter into ordinary computations at all. It would seem that equally simple methods could be applied to the design and analysis of reinforced concrete beams, girders, and floor slabs, without sacrificing any attainable accuracy; not only would this make a given computation more simple, but it would lead to certain principles of economical design, which would render unnecessary any preliminary trial designs, which are otherwise desirable for the sake of economy, where the conditions do not fix dimensions regardless of such considerations.

Several engineers have proposed formulas in which the resisting moment of a reinforced concrete beam is expressed as the integrated

* *Transactions*, Am. Soc. C. E., Vol. LIV, Part E, 1905.

moment of the compressive stresses in the concrete about an axis passing through the steel reinforcement. In some of these the lever arm was assumed as a constant percentage of the depth of the steel below the upper surface of the beam; in others it was a variable, depending upon the elastic properties of the materials, and their respective working stresses. Other engineers have proposed formulas in which the resisting moment of the beam is expressed as the moment of the total stress in the steel about an axis passing through the centroid of the stresses in the concrete, or a point approximating this in position. In some of these, the lever arm is variable, in which case the centroid is accurately located for some assumed form of stress-strain curve; in others, it is assumed as a constant percentage of the depth of the steel below the upper surface, in which case the center of moments does not, in general, coincide exactly with the actual centroid.

The writer proposes a formula of the last mentioned type, in which the lever arm is a constant percentage of the depth, so that, if

A = the total sectional area of steel,

d = the depth of the center of gravity of the steel below the upper surface,

h = a constant,

T = the unit stress in the steel (assumed to be uniform over the cross-section), and

M = the resisting moment of the beam,

then $M = h\,d\,A\,T$ ..... (0)

In any formula of this type, A must be expressed as some fraction of the area of concrete, or some other means must be adopted to avoid an excessive proportion of steel. It is one of the merits of the more complicated theoretical formulas that they contain the unit stresses in both the concrete and the steel, and thus automatically regulate the percentage of steel. But these unit stresses are deduced mainly from experiments on small and isolated specimens of the two materials. Physical properties deduced from the results of such tests can undoubtedly be used as a guide to safe designs; but the writer believes that more accurate, and therefore safer and more economical, results could be secured by numerous tests on beams made up with varying qualities of concrete and varying percentages of steel, with a view to determining the value of the con-

stant, h, in Equation 0, and of the maximum allowable values of A, in terms of the area of concrete, for various grades of concrete and steel. From the results of such experiments, a table of values could be formed, which, used in connection with Equation 0, would greatly simplify the work of design, without the sacrifice of any degree of accuracy attainable by the use of the more complicated formulas.

In the absence of such experiments, however, good working values for h, and for the allowable percentage of steel, can be deduced from the more complicated formulas. Some discussion of these formulas is essential to the objects of this paper, and as this extends to the method by which they are deduced, this deduction, with some modifications, is repeated herein from the writer's paper presented before the International Engineering Congress of 1904.

The point of first importance, in this connection, is the form of the stress-strain curve of concrete, in compression. This has been assumed to be a right line, a parabola, or an empirical curve. So many tests of concrete in compression are available that the writer prefers to assume an empirical curve, determined from the average results of many tests. Among recorded tests, probably none are more accurate than those reported in "Tests of Metals, etc.," at the Watertown Arsenal, for various years. The writer has spent a good deal of time in studying these tests, beginning with those in the report of 1898. Several conclusions seem to be warranted, as follows:

1.—The modulus of elasticity of concrete—or at least its resistance to deformation—increases somewhat with the ultimate strength.

2.—The modulus of elasticity—or the resistance to deformation—increases quite rapidly with the richness of the mixture.

3.—The stress-strain curve for very rich mixtures, especially rich mortars, does not differ much from a right line; but for leaner mixtures, including all those commonly used in practice, it differs very materially from a right line.

4.—In combining the results of many tests, in order to determine the average or true form of the stress-strain curve, it is necessary to combine tests in which the ultimate strength is about the same.

Most of the writer's studies and combinations were in connection with the tests of 1898 and 1899, in which the gauged length of

the specimen, as a rule, was only 5 in.; but other tests, in which a greater gauged length was used, up to 50 in., recorded in later reports, including that of 1904, only confirm the conclusions deduced from the earlier tests, when the richness of the mixture, the ultimate strength, etc., are all considered.

In deducing the constants which will hereafter be used in this paper, seventy-seven apparently normal tests from the reports of 1898 and 1899 were selected. Failure occurred in all these at from 2 500 to 3 500 lb. per sq. in. The tests were first grouped according to ultimate strength, and these groups were sub-divided according to the age and composition of the concrete. Altogether, twelve groups were thus obtained; the elastic compression, as deduced from the record—*i. e.*, the total deformation less the set—was taken as the observed quantity in each case. The arithmetical means of the observed quantities were taken for each group; with these as one set of ordinates, and the applied loads, in pounds per square inch, as the other set, a curve was plotted for each of the twelve groups of tests. All twelve curves differed appreciably from a right line, indicating a rather rapidly decreasing modulus of elasticity under increasing loads. The area enclosed by the curves and the axes of co-ordinates was from one-fifth to one-fourth greater than that included between the same axes and a right line passing through the initial and final points of the curves.

With a stress-strain curve of this sort, it seems better and more accurate, in deducing formulas for the design of reinforced concrete beams, to assume a certain definite percentage of the ultimate strength of the concrete as the maximum stress to be reached when the stress in the steel is at some critical point such as the elastic limit. In designing, it is only necessary to multiply the working load by the desired factor of safety, based on the elastic limit of the steel, and then determine the section so that, under the multiplied load, the assumed maximum stresses shall not be exceeded. The actual working stress in the concrete would seem to be of secondary importance, as long as the factor of safety is assured. Designing for working stresses is accurate only where the modulus of elasticity is constant—under such circumstances, the result is the same as by the method of multiplied loads. In all other cases, the writer thinks the latter method is to be preferred.

It has been not uncommon to assume the elastic limit of the steel and the ultimate strength of the concrete as the corresponding maximum stresses. This practically assumes that failure by rupture or collapse will result when the stress in the steel reaches the elastic limit, no matter what the stress in the concrete may be. No doubt this assumption is quite correct for most methods of reinforcement. It probably leaves some margin of safety in the concrete, for it seems probable that, under the conditions existing in practice, the compressive modulus of rupture of the reinforced concrete is somewhat greater than the ultimate strength of plain concrete in direct compression.

The writer, however, believes that it is possible to design the reinforcement so that total failure need not occur when the stress in the steel reaches the elastic limit; of course, permanent deformation, necessitating ultimate renewal, will undoubtedly occur, just as in the case of a rolled beam in which the stress exceeds the elastic limit; but there is no apparent reason why total collapse should follow, in either case, if the concrete is completely reinforced. For this reason, it seems better to take a conservative figure—say 80% of the ultimate strength—as the maximum stress in the concrete when that in the steel reaches the elastic limit. All the deductions which follow are based on this assumption.

After plotting the twelve curves referred to, the writer measured the area enclosed by the axes of co-ordinates and that part of each curve included between the initial point and the point corresponding to 80% of the ultimate strength. The results were quite interesting; the areas varied from 55.2% of the rectangle enclosed by the axes and the co-ordinates of the 80% point, to 60.3% of the same rectangle, the average of seventy-seven tests being 57.2 per cent. It seems justifiable, therefore, to assume 0.57 as the area factor for the 80% portion of the stress-strain curve of concrete in compression. Several approximate but fairly accurate determinations indicated that the center of gravity of the above area could be assumed at a distance equal to 64% of the 80% ordinate, above the axis of stress, *i. e.*, the axis corresponding to the neutral axis in an actual beam. With these quantities determined, it is possible to deduce actual working formulas. It is assumed that the beam is horizontal, and the applied forces vertical; that, within the elastic

limit of the steel, sections plane before deflection remain plane after deflection; that there is such a union of steel and concrete as to make them act together; that there are no initial strains, and that the steel takes all the tensile stress, the tensile strength of the concrete being neglected.

Let E_s represent the modulus of elasticity of steel,
E_c, the modulus of elasticity of concrete,
T, the unit tensile stress in the steel,
t_s, the elastic limit of steel,
F, the unit compressive stress in the concrete,
f_c, the ultimate strength of concrete in compression; all in pounds per square inch.

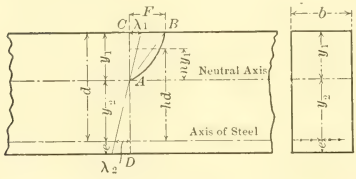


FIG. 1.

Referring to Fig. 1:

Let b represent the width of a rectangular beam or of a part of a floor slab, in inches.
a, the sectional area of steel per inch of width,
$a\,b$, the total sectional area of steel in the width, b,
y_1, the distance of the neutral axis from the extreme element in compression,
y_2, the distance of the neutral axis from the axis of the steel reinforcement,
λ_1, the compression per unit of length in the extreme upper element of concrete,
λ_2, the elongation per unit of length in the steel (assumed as uniform, and the same as that at the axis of the steel, since the cross-section of the bars is relatively small),

d, the distance from the extreme element in compression to the axis of the steel,

$n\,y_1$, the distance from the centroid of the compressive stresses to the neutral axis,

$h\,d$, the lever arm of the stresses in the steel about the centroid of the compressive stresses, and

$A\,B$, the stress-strain curve of concrete in compression, such that when F, for the extreme element in compression, is equal to $0.8\,f_c$, $n\,y_1 = 0.64\,y_1$, and the area, $A\,B\,C = 0.57\,F\,y_1 = 0.456\,f_c\,y_1$.

The following equations are easily deduced:

$$y_1 + y_2 = d \quad \ldots\ldots (1)$$

$$\frac{\lambda_2}{\lambda_1} = \frac{y_2}{y_1}\,;\ \lambda_1 = \frac{F}{E_c}\,,\ \text{and}\ \lambda_2 = \frac{T}{E_s}\,.$$

Therefore, $$\frac{y_2}{y_1} = \frac{T\,E_c}{F\,E_s},\ \text{and}\ y_2 = \frac{T\,E_c}{F\,E_s}\,y_1 \quad \ldots\ldots (2)$$

If $F = 0.8\,f_c$ when $T = t_s$, $y_2 = \dfrac{t_s\,E_c}{0.8\,f_c\,E_s}\,y_1 \quad \ldots\ldots (3)$

Equation 2 applies as long as T does not exceed t_s; but the values of F and E_c must always be those that correspond to each other.

Assuming $F = 0.8\,f_c$ and $T = t_s$, the conditions of equilibrium give,

$$0.456\,b\,f_c\,y_1 = a\,b\,t_s \quad \ldots\ldots (4)$$

If M represents the moment due to the applied forces,

$$M = 0.456 \times 0.64\,b\,f_c\,y_1^{\,2} + a\,b\,t_s\,y_2 = 0.292\,b\,f_c\,y_1^{\,2} + a\,b\,t_s\,y_2 \quad \ldots\ldots (5)$$

It is really not a very complicated operation to use the above equations for designing. They are based on the assumption that both materials are to be worked up to the limit of safety. It is only necessary to substitute proper values of f_c, t_s, E_s and E_c, depending upon the materials used, and the value of M determined from the conditions of loading. In determining M, the working load should be multiplied by the factor of safety.

Testing a given design, however, not made in accordance with the assumptions underlying the above formulas, is not so simple. Probably several approximations, involving a knowledge of the complete form of the stress-strain curve, and changes in the constants of the formulas, would be required to give reliable results.

Consideration of the figure and the equations will show that the coefficient, h, of the lever arm, $h\ d$, does not vary a great deal for widely varying stresses in the steel and the concrete, nor for widely varying positions of the neutral axis. It will be less under heavy stresses, as when the stress in the steel is at the elastic limit, than under ordinary working stresses. As a matter of safety, therefore, if it is to be used as a constant, its value should be determined for the greater stresses.

Assuming that h is a constant, the value of which is to be determined from conditions existing when the stress in the steel is at the elastic limit—*i. e.*, when $T = t_s$, it is possible to write the equation,

$$M = h\ d\ a\ b\ t_s \qquad (6)$$

which is the same in form as that proposed in the earlier part of this paper. (Equation 0.)

To show the variation in the values of h, the following results, deduced from Equations 1 to 5, are submitted, f_c being assumed at 2 500, and $\frac{E_c}{E_s}$ at $\frac{1}{15}$:

If $t_s =$	35 000	lb.	per sq. in.,	$h = 0.834$
If $t_s =$	40 000	"	" "	$h = 0.846$
If $t_s =$	45 000	"	" "	$h = 0.856$
If $t_s =$	50 000	"	" "	$h = 0.865$
If $t_s =$	55 000	"	" "	$h = 0.873$
If $t_s =$	60 000	"	" "	$h = 0.880$
If $t_s =$	100 000	"	" "	$h = 0.920$

Of the various forms that have been assumed for the stress-strain curve, the right line will give the greatest value of h, and a parabola with its vertex on the neutral axis will give the least. Taking an average case from those given above, as for $t_s = 45\ 000$ lb. per sq. in. (f_c and $\frac{E_c}{E_s}$ remaining the same) gives:

For the right line, $h = 0.865$

" " parabola, $h = 0.84$

It would seem justifiable, from the above values, to assume a constant value of 0.85 for h, under all conditions.

Equation 6 can be used just as well for working stresses as for maximum stresses, by substituting T for t_s.

In order to complete Equation 6, $a\ b$ must be expressed in terms of $b\ d$, or, what is the same thing, a must be expressed in terms of d, to insure the use of the correct proportion of steel. The writer thinks that the maximum allowable percentage of reinforcement, for various qualities of steel and of concrete, would best be determined by direct tests. It is certain that it can never be economical to use enough steel to cause the beam to fail first by crushing the concrete; it may be economical to use much less; so that the maximum allowable percentage of steel for each grade of concrete is an important figure to determine, and it should be determined as accurately as possible, by experiments devised with that end in view.

In the absence of better sources of information, this percentage can be determined from formulas such as Equations 1 to 5. Making the same assumptions as for the value of h, these formulas give the following results:

When $t_s = 35\,000$, $\frac{a}{d} = 0.015$, whence $a = 0.015\ d$.

" $t_s = 40\,000$, $\frac{a}{d} = 0.0122$, " $a = 0.0122\ d$.

" $t_s = 45\,000$, $\frac{a}{d} = 0.0101$, " $a = 0.0101\ d$.

" $t_s = 50\,000$, $\frac{a}{d} = 0.00855$, " $a = 0.00855\ d$.

" $t_s = 55\,000$, $\frac{a}{d} = 0.00731$, " $a = 0.00731\ d$.

" $t_s = 60\,000$, $\frac{a}{d} = 0.00633$, " $a = 0.00633\ d$.

" $t_s = 100\,000$, $\frac{a}{d} = 0.00263$, " $a = 0.00263\ d$.

Other assumptions as to the form of the stress-strain curve would result, of course, in different values of $\frac{a}{d}$. Thus, for the assumption of a right line, when $t_s = 45\,000$, $\frac{a}{d} = 0.0089$, whence $a = 0.0089\ d$.

For the assumption of a parabola, when $t_s = 45\,000$, $\frac{a}{d} = 0.0118$, whence $a = 0.0118\ d$.

It will readily be seen from the above that the different assumptions as to the form of the stress-strain curve will give different

depths for a beam of given width, to develop a given resisting moment. The writer thinks the right line gives beams heavier than necessary, and that the parabola rather inclines to the opposite extreme. Thus, if the desired resisting moment per inch of width is 25 000 inch-pounds, if $f_s = 45\,000$, and other constants are as assumed above, the right line would give $d = 8.5$ in., the writer's assumption would give $d = 8$ in., and the parabola would give $d = 7.5$ in.

It may be objected that the approximations introduced in a formula, such as Equation 6, make it less accurate than Equations 1 to 5, or others of the same form deduced from other assumptions as to the stress-strain curve; but a careful examination of the quantities entering Equations 1 to 5, and the method of determining their values, will soon disclose sources of error much greater than any of the approximations introduced into Equation 6. Thus, it requires but the plotting of a few stress-strain curves, from tests of such concrete as is actually used in practice, to demonstrate that, whatever else they are, they are neither right lines, nor very close approximations thereto. Whether the assumption of a parabola, or the assumptions made by the writer, are much nearer the truth, is not a matter of much consequence, when a little consideration is given to the quantity, E_c. This, in the writer's deductions, and all similar ones that he has seen, is called the "modulus of elasticity of concrete;" but, as pointed out by W. K. Hatt, Assoc. M. Am. Soc. C. E., in the discussion of the writer's paper presented at St. Louis, and previously referred to, before using this term, it ought to be defined. As used in Equations 1 to 5, it is undoubtedly a quantity proportional to the tangent of the angle, $B\ A\ C$, in Fig. 1; but it is doubtful whether it is fair to call this a modulus of elasticity, in the same sense as that term is used when applied to materials having a well-defined elastic limit. It has become customary to determine, from tests of concrete, the value of E_c from say 0 to 600 lb. per sq. in.; from 600 to 1 000; from 1 000 to 1 500, and so on. Such a value of E_c is proportional to the tangent of the angle made by a certain chord of the curve, $A\ B$, with the axis, $A\ C$. It would certainly not be correct to use it in formulas such as Equations 1 to 5, unless the design is based on working stresses, and the value of E_c is determined simply between 0 and the working stress. Prob-

ably a more strictly accurate value of E_c would be proportional to the tangent of the angle made by the curve itself with the axis, $A\ C$. This, however, would not be properly applicable in any formulas the writer has seen, up to the present time; but, asuming that E_c shall be, as in Equations 1 to 5, proportional to the tangent of the angle, $B\ A\ C$, it will soon be found that it will vary within such wide limits that an average value assumed in calculations may easily vary from the actual value by at least as much as 25 per cent. This would introduce a very material error in both the percentage of reinforcement and the probable maximum stresses in the concrete. Therefore it seems justifiable to assume that a formula such as Equation 6 can be made just as accurate as any of the forms of Equations 1 to 5, especially if its constants be determined from tests designed with that end in view.

There are other advantages in the form of Equation 6, however, besides the most apparent one, of simplicity. If the maximum allowable percentage of steel is correctly determined for the qualities of steel and concrete it is proposed to use in each case, Equation 6 will give a design in which both materials are worked to the safe limit; this design, from the standpoint of efficient use of material, will be the most economical. If the constants entering Equations 1 to 5 are correctly determined, they will give similar results. This kind of economy, however, is not necessarily equivalent to minimum first cost, in dollars and cents. With Equations 1 to 5, several tentative designs, with estimates of cost, would be required to determine the one of minimum cost. This results from the fact that the ratio of cost of steel per cubic foot to the cost of concrete per cubic foot is entirely independent of the ratio between the maximum allowable stresses of the two materials. It is desirable to express the cost in terms of the first-named ratio, and then to apply the principles of maxima and minima, if possible, so as to disclose at once the conditions leading to minimum cost. With Equations 1 to 5, this cannot be done, because, as soon as the materials are selected, their physical qualities fix the percentage of steel, which is a constant from that moment. Thus, from Equation 3 it is seen that the coefficient of y_1 is made up of quantities, all of which are physical constants, peculiar to the materials. Therefore, as soon as the materials are selected, the ratio of y_1 to y_2 is a con-

stant, whence the ratio of y_1 to d is a constant. From Equation 4, it is seen that $\frac{a}{y_1}$ becomes a constant, as soon as the physical constants are determined, therefore, $\frac{a}{d}$, or the proportion of steel, is also a constant, under the same conditions.

Now assume a beam of unit width, for simplicity; let the moment of the applied loads for a beam of unit width be represented by m. Making the necessary substitutions in Equation 6, it becomes,

$$m = h\, d\, a\, t_s \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

or

$$a = \frac{m}{h\, d\, t_s} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which a and d are the variables, since m is constant for any given case.

The cost of a concrete beam may be subdivided into:

(1) The cost of centering,

(2) the cost of the concrete below the steel,

(3) the cost of the concrete above the steel, and

(4) the cost of the steel itself.

(1) and (2) are fixed by the conditions of each case, and would not vary appreciably for variations in $\frac{a}{d}$; they may, therefore, be treated as constants. The variable part of the cost then lies in (3) and (4). The cost of the steel is proportional to a, and that of the concrete to d.

Let p represent the ratio: $\frac{\text{Cost of steel per cubic foot}}{\text{Cost of concrete per cubic foot}}$.

Let x represent a quantity proportional to the sum of the costs of the variable elements, so that

$$x = p\, a + d \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Substitute for a its value from Equation 8, and there results,

$$x = \frac{p\, m}{h\, t_s\, d} + d \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

whence,

$$h\, t_s\, d\, x = p\, m + h\, t_s\, d^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Differentiating,

$$h\, t_s\, (d\, \delta\, x + x\, \delta\, d) = 2\, h\, t_s\, d\, \delta\, d \ldots\ldots\ldots\ldots\ldots (12)$$

Whence,

$$\frac{\delta\, x}{\delta\, d} = \frac{2\, d - x}{d} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

For a minimum, $\frac{\delta x}{\delta d} = 0$, whence

$$2d - x = 0, \text{ or, } x = 2d \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

It is, therefore, a condition of minimum cost, that the total variable cost shall be twice that of the variable concrete, or, in other words, the cost of the steel and the cost of the variable part of the concrete must be equal.

Substitute $2d$ for x, in Equation 9, and there results,

$$a = \frac{d}{p} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

i. e., for minimum cost, the area of steel must be equal to the area of concrete above it, divided by the quantity, p.

If l be the span of the beam, in inches, and w the total load per linear inch, in pounds, $m = \frac{w l^2}{8}$.

Substitute this value of m, and the value, $\frac{d}{p}$, for a, in Equation 7, and there results,

$$\frac{w l^2}{8} = \frac{h t_s d^2}{p} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

whence, $$d^2 = \frac{p w l^2}{8 h t_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

In the above demonstration, m, the total moment, has been assumed as constant. As a matter of fact, the dead load varies with both a and d. The variation due to variations in a is very small, and can be neglected; that due to variations in d is not negligible, however, and if it is desired to provide an absolutely constant live-load capacity, the total moment must be expressed in terms of d as a variable. The weight of the concrete below the steel must be included as part of the constant live load, in this case.

Let w' represent the weight of concrete, in pounds per cubic inch,

" M_1 represent the total bending moment on a beam of unit width,

" w represent the constant live load, per linear inch,

" m represent the moment due to the constant live load,

" m' represent the moment due to the variable dead load.

$$M_1 = m + m' = \frac{w l^2}{8} + \frac{w' d l^2}{8} \quad \ldots\ldots\ldots\ldots\ldots (18)$$

$$a = \frac{M_1}{h\ t_s\ d} = \frac{w\ l^2}{8\ h\ t_s\ d} + \frac{w'\ d\ l^2}{8\ h\ t_s\ d} \quad \ldots\ldots\ldots\ldots (19)$$

Whence,

$$x = d + p\ a = d + \frac{p\ w\ l^2}{8\ h\ t_s\ d} + \frac{p\ w'\ d\ l^2}{8\ h\ t_s\ d} \quad \ldots\ldots\ldots\ldots (20)$$

$$8\ h\ t_s\ d\ x = 8\ h\ t_s\ d^2 + p\ w\ l^2 + p\ w'\ d\ l^2 \quad \ldots\ldots\ldots\ldots (21)$$

Differentiating,

$$\frac{\delta\ x}{\delta\ d} = \frac{-\ 8\ h\ t_s\ x + 16\ h\ t_s\ d + p\ w'\ l^2}{8\ h\ t_s\ d} \quad \ldots\ldots\ldots\ldots (22)$$

For a minimum, $-\ 8\ h\ t_s\ x + 16\ h\ t_s\ d + p\ w'\ l^2 = 0$

whence, $$x = 2\ d + \frac{p\ w'\ l^2}{8\ h\ t_s} \quad \ldots\ldots\ldots\ldots (23)$$

But, $x = d + p\ a$, whence,

$$a = \frac{d}{p} + \frac{w'\ l^2}{8\ h\ t_s} \quad \ldots\ldots\ldots\ldots (24)$$

The term, $\frac{w'\ l^2}{8\ h\ t_s}$, in Equation 24, is the area of steel required to carry the variable dead load alone, and the term, $\frac{p\ w'\ l^2}{8\ h\ t_s}$, in Equation 23 is proportional to the cost of this steel. It is to be noted that the quantity of steel required to carry the variable dead load is, for a given span, a constant, and independent of both the live load and the depth, d.

Evidently, the total cost is a minimum when it is equal to twice the cost of the variable concrete, plus the cost of the steel required to carry the variable dead load—the latter item of cost being constant for a given span, under all conditions of loading.

Equating the second members of Equations 19 and 24, and solving for d^2, there results,

$$d^2 = \frac{p\ w\ l^2}{8\ h\ t_s} \quad \ldots\ldots\ldots\ldots (25)$$

This equation is identical in form with Equation 17. $\frac{w\ l^2}{8}$ is the moment due to the live load. It is evident, therefore, that the economical value of d is dependent solely upon the live load, for a given value of l. Unless the live load is very small, the values of x given by Equations 14 and 23 do not differ by appreciable amounts, especially if the design is made for maximum stresses, the load producing the maximum stresses being computed as the dead load, plus

the product of the working live load by a factor of safety; this, after all, is the more logical plan; what is wanted is a factor of safety under applied loads. If the dead load is added to, say, four times the working live load, and the total taken as the load producing maximum stresses, the factor of safety under total working loads is quite as great as it is in any ordinary fire-proof building design, and, for static loads, is certainly sufficient. Under such assumptions, the results of Equations 14 and 23 do not differ by more than $\frac{1}{3}$ of 1 per cent.

If, in Equation 19, $\frac{d}{p}$ is substituted for a, and the resulting equation solved for d, there results a value, which, while not theoretically the most economical, will require the cost to be computed to tenths or possibly hundredths of cents per linear foot of a beam 12 in. wide, to show the difference. This is close enough to theory for all practical purposes, and is the method the writer would recommend, where even great accuracy is desired.

Evidently, the value of the quantity, p, is of some importance; probably a fair average value of concrete per cubic foot, apart from the centering, is 20 cents. At 3 cents per lb., the value of steel is \$14.40 per cu. ft. A fair average value of p, therefore, would be 72, and $\frac{a}{d}$ would then be equal to $\frac{1}{72}$, or 0.014. An examination of the values before deduced for the maximum allowable percentage of steel, will readily show, however, that average values of $\frac{1}{p}$ are too great for steel of high elastic limit, because the concrete would not be able to develop the strength of so much steel. Under these circumstances, the most economical practicable design is the one using the maximum allowable percentage of steel; theoretical economy, based on relative costs, is not attainable. The best that can be done is to use as large a percentage of steel as the concrete will stand; any more than this is wasted, for when the concrete fails, the beam fails. Since the foregoing discussion of minimum cost does not contain the ratio between the allowable maximum stresses in the two materials, it is but natural that the results should require comparison with those derived from formulas in which this ratio is a factor. In actual design, the writer would use the value, $\frac{d}{p}$ for a,

unless the maximum allowable value of a, as deduced from experiment or from Equations 1 to 5, should be less than $\frac{d}{p}$; in which case the maximum allowable value of a would be used. He would, if great accuracy is desired, write the moment of the loads in the form given in Equation 18, in order to avoid any guessing as to the dead load, due to the variable part of the concrete; w would be the total live load, multiplied by the factor of safety.

The only equation required for designing would then be Equation 19. For a, would be substituted its proper value, in terms of d, and the equation would be solved for d. For a beam of width, b, it is only necessary to make very obvious changes in the formula. The value of a follows, when d is known. If great accuracy is not necessary, an approximate value for the dead load can be assumed;

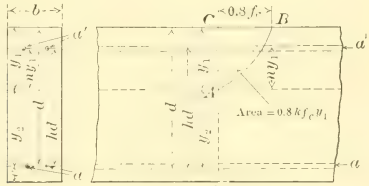


FIG. 2.

the total moment is then a constant, and the equation giving the value of d will be a pure quadratic, instead of an adfected one, as in the other case.

The question is at once suggested by the results above deduced, whether, in the case of steel of high elastic limit, in which the maximum allowable value of a is much less than $\frac{d}{p}$, greater economy could not be attained by reinforcing the concrete against compression. It is of interest, therefore, to determine the conditions of minimum cost when double reinforcement is used.

Let Fig. 2 represent a cross-section, and a partial side elevation, of a concrete beam, reinforced at top and bottom. It would not be desirable to place the top reinforcement much higher than the centroid of the compressive stresses in the concrete; and as this

position makes the calculations simpler, without affecting any fundamental principle involved, it is assumed, for this discussion. The various quantities are as hitherto assumed, in Fig. 1, and the symbols have the same meaning. Conditions producing the maximum allowable stresses are supposed to exist.

From Equation 3, $y_2 = \frac{t_s}{0.8} \frac{E_c}{f_c} \frac{}{E_s} y_1$.

Let $$\frac{t_s}{0.8} \frac{E_c}{f_c} \frac{}{E_s} = g.$$

Then, $y_2 = g\, y_1$ ..... (26)

$$d = y_1 + y_2 = (1 + g)\, y_1.$$

Therefore, $y_1 = \frac{d}{1+g}$ ..... (27)

$$h\, d = (g + n)\, y_1 = \frac{g+n}{1+g} d \text{ .....} (28)$$

Total stress in concrete $= 0.8\, b\, k\, f_c\, y_1 = \frac{0.8\, b\, k\, f_c\, d}{1+g}$.

" " " $a' = a'\, t_s \frac{n\, y_1}{y_2} = \frac{a'\, t_s\, n}{g}$.

" " " $a = a\, t_s$.

$$a\, t_s = \frac{a'\, t_s\, n}{g} + \frac{0.8\, b\, k\, f_c\, d}{1+g} \text{ .....} (29)$$

Assume a constant total moment, thus ignoring the variations in dead load, due to changes in d. Assume that, the materials once selected, h is constant.

Consider a beam in which $b = 1$. a may be divided into two parts, a_1 and a_2, such that $a_1\, t_s$ will be equal to the stress in a', and $a_2\, t_s$ will be equal to the stress in the concrete.

Then, $a_1\, t_s = \frac{a'\, t_s\, n}{g}$, whence $a' = \frac{a_1\, g}{n}$ ..... (30)

Since m, the total moment, is equal to $h\, d\, t_s\, a$, it may be divided into two parts, m' and m'', such that

$$a_1 = \frac{m'}{h\, t_s\, d}, \text{ and } a_2 = \frac{m''}{h\, t_s\, d} \text{ .....} (31)$$

$$a_1 + a' = a_1 + \frac{a_1\, g}{n} = \frac{(n+g)\, a_1}{n} \text{ .....} (32)$$

Substitute for a_1, in the second member of Equation 32, its value from Equation 31, then

$$a_1 + a' = \frac{n+g}{n} \cdot \frac{m'}{h\, t_s\, d} \text{ .....} (33)$$

It is evident that a_2 will be the maximum allowable area of steel, as determined from Equations 1 to 5. The total area of steel, or $a + a'$, will be equal to $\frac{m''}{h\ t_s\ d} + \frac{n+g}{n} \cdot \frac{m'}{h\ t_s\ d}$.

Let x be proportional to the cost of the beam; then

$$x = d + \frac{p\ m''}{h\ t_s\ d} + \frac{n+g}{n} \cdot \frac{p\ m'}{h\ t_s\ d} \quad \ldots\ldots\ldots (34)$$

Whence,

$$n\ h\ t_s\ d\ x = n\ h\ t_s\ d^2 + n\ p\ m'' + (n+g)\ p\ m'.$$

Differentiating,

$$n\ h\ t_s\ (d\ \delta\ x + x\ \delta\ d) = 2\ n\ h\ t_s\ d\ \delta\ d \quad \ldots\ldots\ldots (35)$$

Whence,

$$\frac{\delta\ x}{\delta\ d} = \frac{-x + 2\ d}{d} \quad \ldots\ldots\ldots\ldots\ldots\ldots (36)$$

For a minimum,

$$-x + 2\ d = 0, \text{ whence } x = 2\ d \quad \ldots\ldots\ldots\ldots\ldots (37)$$

Therefore, in this case also, the total cost is a minimum when the cost of the steel is equal to that of the variable part of the concrete.

To determine whether the beam with double reinforcement, complying with this economical condition, is cheaper to resist a given moment than one with single reinforcement, in which the maximum allowable percentage of steel is used, it is necessary to compare the results for some particular grades of steel and concrete. Let $f_c = 2\,500$, and $t_s = 100\,000$.

Then, by Equations 1 to 5, $g = 3\frac{1}{3}$, $h = 0.92$, $\frac{a}{d} = 0.00263$, and $n = 0.64$.

Let $p = 72$, then $\frac{1}{p} = 0.014$. Represent $\frac{1}{p}$ by p'. Let p'' represent the maximum allowable proportion of steel for lower reinforcement only. Then $p'' = \frac{a}{d} = 0.00263$ (when $t_s = 100\,000$).

$a_2 = p''\ d$. $a_1 + a_2 + a' = p'\ d$.

Whence,

$$a_1 + a' = (p' - p'')\ d \quad \ldots\ldots\ldots\ldots\ldots (38)$$

But, from Equation 32, $a_1 + a' = \frac{n+g}{n}\ a_1$.

Equate the second members of Equations 38 and 32, and there results, $\frac{n+g}{n}\ a_1 = (p' - p'')\ d$.

whence,

$$a_1 = \frac{n}{n+q}(p' - p'')d \quad \ldots\ldots\ldots\ldots\ldots\ldots (39)$$

$a = a_1 + a_2$, and $m = h\, t_s\, d\, a$, whence,

$$m = h\, t_s\, d\,(a_1 + a_2) = h\, t_s \frac{n}{n+q}(p' - p'')d^2 + h\, t_s\, p''\, d^2. \quad (40)$$

Substituting for the various constants their values,

$$m = (92\,000 \frac{0.64}{3.97} \times 0.01137 d^2) + (92\,000 \times 0.00263 d^2) = 410 d^2 \ldots (41)$$

$$d^2 = \frac{m}{410}, \text{ and } d = \frac{\sqrt{m}}{20.2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (42)$$

But, $x = d + p\,(a + a') = 2\,d$.

Substitute for d its value, in terms of m, and there results

$$x = \frac{\sqrt{m}}{10.1} = 0.099\ \sqrt{m} \quad \ldots\ldots\ldots\ldots\ldots\ldots (43)$$

If lower reinforcement only is used,

$$m = h\, t_s\, a_2\, d = h\, t_s\, p''\, d^2 = 242\, d^2 \ldots\ldots\ldots\ldots (44)$$

$$d^2 = \frac{m}{242}, \text{ and } d = \frac{\sqrt{m}}{15.5},$$

$$x = d + p\, a_2 = \frac{\sqrt{m}}{15.5} + \left(72 \times 0.00263 \frac{\sqrt{m}}{15.5}\right) = 0.077\ \sqrt{m} \ldots (45)$$

Comparing this result with that of Equation 43, it is evident that, when $t_s = 100\,000$, it is cheaper to use lower reinforcement only. If this is true for this value of t_s, it is certainly true for any other value likely to be used in practice. It is advisable, therefore, to use double reinforcement only when the conditions of the problem restrict the value of d, or when it is very important to reduce the dead weight to a minimum. In the foregoing discussion, the introduction of d as a variable factor in the value of m has not been considered; but there would ordinarily be no material change in the result, if this were done. It is to be noted, however, that when only lower reinforcement is used, and $t_s = 100\,000$, the value of d is about one-third greater than when double reinforcement is used. This will make some difference in the actual live-load capacity, if m is assumed as a constant. If the live load is very small, as compared with the dead load, this may change the relative costs, for a given live-load capacity, so as to make the double reinforcement more

economical; but if the live load is relatively large, the single reinforcement will be more economical, although the difference may not be quite as great as indicated by Equations 43 and 45. In practically all cases that would ordinarily occur, the single reinforcement will be the cheaper, by at least 20 per cent.

Hitherto, it has been assumed that the beam was to be reinforced against what may be termed the flange stresses only, and the discussion applies, of course, to the section where m is a maximum; but other stresses exist in reinforced concrete beams, corresponding to the web stresses in steel beams. For simplicity, they will hereafter be referred to as web stresses. They might be called shearing stresses, for the web is the locus of such stresses, as well as those due to tension and compression; by analogy with an open truss, in which the web stresses in a panel are often called the "shear" in that panel, all the web stresses in a reinforced concrete beam might be called the "shear"; but this use of the term has led to much useless discussion and misunderstanding, and the writer prefers the other, especially considering the fact that if tensile and compressive web stresses are adequately provided for, the shearing stresses proper will also be provided for.

Assuming the beam to be uniformly loaded, the ideal web reinforcement would consist of a multiplicity of relatively small members attached to the horizontal reinforcement, and inclined both ways from the center of the span, at an angle of 45 degrees. The attachment to the horizontal reinforcement should be independent of the concrete, should permit no lost motion between web and flange reinforcement, and should be strong enough to develop the strength of the former. The spacing should vary from the center to the ends, decreasing in proportion to the increase in the web stresses. In any case, the sum of the horizontal components of the stresses in all the web members in each half of the beam should be at least equal to the maximum stress in the flange reinforcement. The web members should all extend entirely to the top of the beam.

Assuming web members as above described, the aggregate cross-section of all on one side of the center is, for a beam of unit width, with horizontal reinforcement, a, equal to $\sqrt{2}\, a$. The length of each web member is equal to $\sqrt{2}\, d$. The volume of all the web members in the beam is then equal to $2 \times \sqrt{2}\, a \times \sqrt{2}\, d = 4\, a\, d$. The vol-

ume of web members per unit of length of the beam is then $\frac{4\,a\,d}{l}$. This is also the area of a bar equal in weight to all the web members, and equal in length to the span, l. Call this area a''. The cost, x, is then equal to $d + p\,(a + a'')$. Represent the live-load moment by m, and the dead-load moment by m', as in Equation 18, then $a = \frac{m + m'}{h\,t_s\,d}$. Substituting the values of a and a'' in the value of x, there results an equation which can be differentiated as in the discussions that have gone before, the variable, d, entering as a factor in the value of the total moment. It is unnecessary to repeat the mathematical work. The results are as follows:

$$\text{Minimum cost, } x = 2\,d + \frac{p\,w'\,l\,d}{h\,t_s} + \frac{p\,w'\,l^2}{8\,h\,t_s} + \frac{4\,p\,w\,l}{8\,h\,t_s} \quad \ldots\ldots(46)$$

$$d^2 = \frac{p\,w\,l^2}{8\,h\,t_s + 4\,p\,w'\,l} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(47)$$

Designs made in accordance with the foregoing results give costs almost identical with those obtained by designing without reference to web members, and then adding the web members. Thus, for a particular case assumed by the writer and worked out, the cost per linear foot of the variable elements of a beam 12 in. wide, as determined under various assumptions, was as follows:

1.—Designed without reference to web members, and without taking account of the variation in d, web members being afterward added.....30.743 cents

2.—Designed without reference to web members, taking account of variations in d, web members being afterward added.....30.81 cents

3.—Designed in accordance with Equation 47 and the assumptions leading up to it.....30.73 cents

Evidently it is not worth while to consider either web members or variations in d, in designing the section of maximum moment, except that the value of M may be written as a function of d, if it is desired to avoid guessing at the dead weight. After a little experience, however, the designer will be able to guess the dead weight with sufficient accuracy, and in that case he will have, as stated above, only a pure quadratic to solve—otherwise he will have to solve one containing both the first and second powers of d.

T-Beams.

All that has gone before is applicable only to rectangular beams and floor slabs. No extensive system of reinforced concrete floors can be economically designed, however, without the use, either of rolled steel beams, or else of reinforced concrete ribs, forming, together with a portion of the floor slab in each case, what is practically a T-beam.

It is necessary, therefore, to consider the economical design of such sections as this.

Let Fig. 3 represent the cross-section of a T-beam.

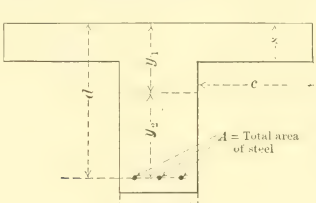


FIG. 3.

The dimension, s, whether the T-beam is isolated, or whether it is incorporated in a floor system, is generally fixed by the conditions of the problem; b is fixed by its relation to s, and c will usually result from b. A few trials may be necessary, however, to get these quantities properly determined. Assume that they have been fixed, and that d is the only variable.

$$M = \frac{w\,l^2}{8} + \frac{w'\,b\,d\,l^2}{8} + \frac{2\,w'\,c\,s\,l^2}{8} \quad \ldots\ldots\ldots\ldots (48)$$

$$x = p\,A + b\,d + 2\,c\,s \quad \ldots\ldots\ldots\ldots (49)$$

By a discussion similar to those that have gone before, it will be found that, for minimum cost,

$$x = 2\,b\,d + 2\,c\,s + \frac{p\,w'\,b\,l^2}{8\,h\,t_s} \quad \ldots\ldots\ldots\ldots (50)$$

$$A = \frac{b\,d}{p} + \frac{w'\,b\,l^2}{8\,h\,t_s} \quad \ldots\ldots\ldots\ldots (51)$$

$$d^2 = \frac{p\,w\,l^2 + 2\,p\,w'\,c\,s\,l^2}{8\,h\,t_s\,b} \quad \ldots\ldots\ldots\ldots (52)$$

If M had been assumed as a constant, these equations would have been

$$\left.\begin{aligned} c &= 2\,b\,d \\ A &= \frac{b\,d}{p} \\ d^2 &= \frac{p\,M}{8\,b\,t_s\,b} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (53)$$

As $b\ d$ is the variable concrete area, it may again be assumed that the economical value of A is $\frac{b\ d}{p}$. The last term in Equation 51 is the area of steel required for carrying simply the variable dead load. If web members be considered, the result would be the same as in the case of rectangular beams.

It is evident, that if $\frac{1}{p}$ does not exceed the value of $\frac{a}{d}$, as deduced from Equations 1 to 5, the economical design for T-beams will be one in which the value of c is zero—*i. e.*, in which there is no projecting flange, and the beam is reduced to a rectangular section. But if $\frac{1}{p}$ is greater than $\frac{a}{d}$, the compressive resistance of a flange, or of a reasonable width of floor slab, may be counted on, and probably, for any grade of steel likely to be used, A could be made equal to $\frac{b\ d}{p}$, in all cases. It will cost a little more to put concrete into the forms for the ribs or stems, so that p will be less, and $\frac{1}{p}$ greater, for T-beams than for floor slabs.

In the writer's St. Louis paper, he presented formulas for T-beams modeled after those deduced by A. L. Johnson, M. Am. Soc. C. E. In applying these formulas to particular cases, the total allowable width of flange ($b + 2\ c$, in Fig. 3) was generally found to be between 3 b and 4 b. The writer thinks conservative practice would confine this width to 3 b. By the assumptions as to the stress-strain curve assumed in this paper, and those made in the St. Louis paper as to the distribution of stresses in the flange of the beam, the average stress in the compressed part of the concrete of a T-beam may be taken as not less than $0.55 \times 0.8\ f_c = 0.44\ f_c$, under the loads producing the maximum stresses assumed in de-

ducing the formulas. If, then, the product obtained by multiplying the area, $b\,y_1 + 2\,b\,s$, by $0.44\,f_c$, is equal to, or greater than, $\frac{b\,d\,t_s}{p}$ the economical value of A can be used. Otherwise, the following equation must be satisfied (c being assumed equal to b),

$$A\,t_s = 0.44\,f_c\,b\,(y_1 + 2\,s) \ldots\ldots\ldots\ldots\ldots\ldots (54)$$

The writer believes that, in the foregoing discussion, the principles of economic design have been brought out; and that the discussion is based on formulas capable of giving as great accuracy as is attainable with any more complicated ones. With the assistance of a few constants, which ought to be determined by specially devised experiments, the formulas herein suggested would greatly simplify the work of designers.

All that has gone before applies mainly to the economical design of the section where the moment is a maximum, and the formulas take account only of bending stresses, pure and simple, as far as they are actually a guide in designing.

The web stresses, in practice, are quite as important as the flange stresses. The web of a plate girder and the web members of a bridge truss often demand much more attention than the flanges and chords. This is equally true of reinforced concrete girders and beams. If it is bad practice to count upon the tensile strength of the concrete in the lower part of the beam, it is equally bad practice to count upon it in the web. Of the existence of tensile stresses in the web, acting at various angles with the axis, there can be no doubt. In Rankine's Applied Mechanics* is given a diagram showing the lines of principal stress in a homogeneous beam. The diagram would not apply to a reinforced concrete beam, but there would be one more or less similar to it, with the lines of tensile stress all originating in the axis of the steel reinforcement of the lower flange, rising at an agle of approximately 45°, and inclined toward the abutments. This would be the case under the assumption of a uniformly or symmetrically distributed load. These lines of tensile stress would not differ much in relative arrangement from the tensile web members of a lattice girder.

About August, 1904, the writer had to design some rather important long-span girders for the War College Building, at Wash-

* Edition of 1895, page 342, Fig. 146.

ington Barracks, D. C. He had, nearly two years before, become convinced that not only ought reinforced concrete beams and girders to be designed with web reinforcement, but that the tensile web members ought to be arranged at an angle of 45°, and firmly attached to the horizontal reinforcement, without the possibility of lost motion, so that the horizontal components of the tensile web stresses could be transmitted into the horizontal reinforcement, without depending on the surrounding concrete; in other words, that concrete should no more be relied upon for connections than for resisting tensile flange stresses. Previous to the time when the War College work was designed, the writer had analyzed the web stresses by analogy with a Pratt truss; but study of the War College problem made it apparent that the

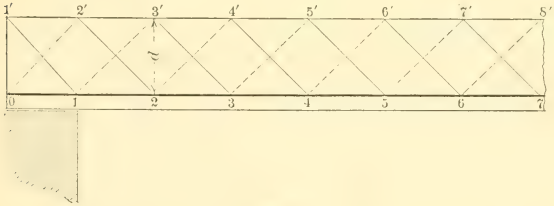


FIG. 4.

double intersection Warren girder was a better analogy, and the writer has used it since that date. To illustrate the method, and to make certain subsequent references more clear, let Fig. 4 represent a longitudinal section of a reinforced concrete beam, taken through the axis of the horizontal reinforcement, only one-half of the span being shown. The beam is divided, as nearly as possible, into equal panels, each panel length, as 1′–2′, or 1–2, being equal to the depth, *d*, of the axis of the horizontal reinforcement below the top of the beam. The lines 1′–1, 2′–2, etc., are the assumed lines of tensile web stresses, and the corresponding compressive stresses in the concrete are assumed to act along the broken lines, 2′–0, 3′–1, etc. This gives a double system of web members, and the web stresses are divided between the two systems, on the assumption that the load is concentrated at panel points, such as 0, 1, 2, 3, etc., or at 2′, 3′, etc. The stress in, say, 2′-2 having been determined, enough web members are

inserted between the middle points of 1–2, and of 2–3, to carry it. This method is followed for all the tensile web stresses. The concrete is assumed to take care of the compression. The tensile web members are designed so that the adhesion of that part above the neutral axis will develop the strength of the member; this is to be insured by the use of a mechanical bond in the upper part of the web member, if necessary. All web members extend quite to the upper surface of the beam.

If the web members hold, mechanical bond for the main horizontal member is probably of secondary importance; but, incidentally, it is furnished in a very efficient way by the attachment of the web members. The writer prefers not to depend upon simple adhesion, if mechanical bond is practicable, though he was not always of this opinion.

To provide against concentrated and rolling loads, some of the web members, near the center of the span, should incline toward the farther abutment, parallel to the lines, 7′–5, 8′–6, etc., so as to act as counterbraces. Just how far in this direction the designer shall depart from the conditions due to a uniformly distributed load is a matter of judgment, in each particular case. For railroad structures, the worst conditions for the web members can probably be determined with a fair degree of accuracy. For other structures, and especially for buildings, some fair average concentrated loads should be assumed, and the web members designed for them, as well as for a uniform load. The lower flange reinforcement should be designed for a uniformly distributed load; it should be diminished in cross-section only very near the abutments, if at all. If designed in this way, it will take care of any pure bending stresses likely to occur, under any reasonable distribution of the load.

The writer is well aware that many engineers will not agree with him that attached web members are necessary; but at least all will admit that the subject is still an open one, and that the question of web stresses, including the shear, is not yet satisfactorily settled.

The writer thinks that nearly all tests of reinforced concrete beams without web reinforcement, in which the percentage of steel was anywhere near its economical value, have shown that failure occurred under the web stresses primarily. This also will be disputed; but, even where the beam was tested by a center load, the

failure has often been by diagonal cracks along the lines of compressive web stress, at greater or less distances from the center.

Failures under uniform loads, in practice, have nearly always been by diagonal cracks near the abutments. In nearly every case, unless the percentage of steel was very small, failure of the beam has occurred when the stress in the steel reached the elastic limit. Tests in which an excessive percentage of steel resulted in crushing the concrete are not here considered. As far as the writer is aware, no thoroughly scientific tests have been made to determine the value of web members such as those herein advocated; but, in the excellent work on Reinforced Concrete by Buel and Hill, at the bottom of page 47 and the top of page 48, are described two tests which indicate very clearly that much good is to be expected from such reinforcement. The owners of a certain patented system, in which attached diagonal web members are used, have made many tests of actual experimental structures, under more or less uniform loadings, and these tests indicate very clearly the value of the web members. In a recent one, the percentage of steel was, as nearly as the writer can determine, more than $1\frac{1}{2}$; failure occurred by pulling the steel in two, under a stress of at least 55 000 lb. per sq. in. This is a very remarkable result. The same concern, in earlier tests, not only pulled the steel in two, but developed a capacity for deflection before rupture, as great as could reasonably be expected of a rolled steel beam. As far as the writer is aware, such tests are the only recorded ones which bear directly on the point at issue. Some other tests have been made by other persons quite accurately, on beams reinforced with the patented bars above referred to, but they were designed so that the entire bar, web members and all, was entirely below the neutral axis of the beam. Naturally, they behaved in much the same way as beams without web members.

Apart from the very positive results of the few tests that have been made, however, there are other reasons for predicting the value of web members, from the results of tests made without them.

It is evident that the sum of the horizontal components of the stresses in the web members of Fig. 4 must be, in each half of the span, at least equal to the total stress in the horizontal reinforcement. Now, suppose there are no web members; the stress must then be transmitted directly between the horizontal reinforcement and the

concrete; the concrete immediately around the reinforcement thus becomes the locus of a great many stresses, both direct and shearing; these are much greater in intensity near the abutments than they are near the center, but, in a general way, the sum total of these stresses is distributed from the center to the abutment. As the stress in the steel approaches the elastic limit, the adhesion of the steel in the concrete near the center of the span grows less and less, due to the diminution in the cross-section of the reinforcing bars; the stresses in the concrete are concentrated more and more toward the abutments, and become, not a question of the total vertical shear, but of the total stress in the steel, multiplied by some such factor as the secant of 45 degrees. Under these circumstances, it is but natural that failure should occur when the stress in the steel reaches the elastic limit, and with greater or less suddenness. The exact locus of the first fatal crack is no doubt determined by accidental variations in the concrete, by shrinkage stresses, or other unforeseeable circumstance. As a rule, in all these tests, the total vertical shear divided by the area of cross-section of the concrete is a very moderate quantity; similarly, this shear, divided by the cosine of 45°, may be taken as the total tensile stress along such a line as 1′–1, and the area of concrete to resist it may be taken as that cut out by a plane passing through the line, 0–2′, and perpendicular to the plane of the figure. This will also give a very moderate unit tensile stress in the concrete, and will not indicate any urgent need of web reinforcement. But when the question of getting the total stress into the steel is considered, as above, it is a very different matter; moreover, in actual structures, because of their great extent, the effects of temperature changes and shrinkage stresses are much more marked than in a single beam made for testing in a laboratory. Probably at the very point where shearing and tensile strength in the concrete are most needed, they will be wholly lacking, although the fact may not be apparent under light loads, due to the binding action of the steel. On the other hand, if there is adequate reinforcement against all possible tensile stresses, none of these practical troubles will, to any appreciable extent, diminish the compressive value of the concrete; and shearing stress, as such, need not be considered, except in reference to the power of the concrete to hold on to the various steel members when they are under stress. Here, again, if the web mem-

bers are designed as herein indicated, the greater the stresses in the beam, and the greater the deflection, the tighter will be the grip of the compressed part of the concrete on the upper parts of the web members—so that failure can occur only by actually crushing the concrete, or by pulling the steel apart. It is probable that the web members would also act so as to distribute the stresses more uniformly over the cross-section of the concrete in compression, and thus increase its efficiency. Under such conditions, collapse would never occur at the elastic limit in a well-balanced design. It is evident that the lower part of the concrete may be in a more or less cracked and damaged condition, and still capable of acting, in compression, as a separator between the horizontal reinforcement and the upper compressed part of the beam, thus carrying the compressive web stresses, which are relatively small. As long as this condition obtains, collapse will not occur, except by actual failure of either the compressed flange, or the horizontal reinforcement, as above stated. This leads up to the question of the influence of the form of reinforcement on the fire-resisting qualities of the structure.

It is fairly well established that, under temperatures of from 800° to 1 000° fahr., cement which has set, begins to lose its water of crystallization; as this change progresses, the strength is impaired. If the fire lasts long enough, the lower part of the concrete in a floor system is completely ruined, and generally comes off, exposing the steel; if the fire still continues, collapse, in ordinary designs, speedily follows. The exposure of the steel is undoubtedly greatly accelerated by the existence of severe stresses in the concrete immediately surrounding the steel, in all cases where attached web members are not used. Where they are used, the damaged concrete, having no structural duty to perform, will hang on longer, and at least protect the steel; even after it comes off, the structure will collapse only after the steel becomes hot enough to have its tensile strength seriously impaired.

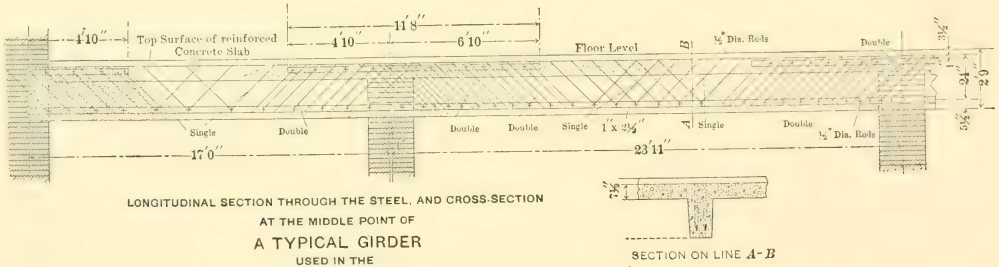
Even if collapse does not occur—in fact, even if the steel is not exposed at all—in cases where attached web members are not used, if the concrete has been dehydrated to any appreciable depth, nothing short of re-building will constitute adequate repairs. The damaged material must come off; on it, and its bond with the concrete above, depended the ability of the steel to transmit its stresses into the

upper part of the beam. There is no known method of replacing the damaged concrete with new material, and of securing a bond between the new and the old, anything like as strong as the original cohesion.

If, however, attached web members are used, there is no necessity, as far as strength is concerned, for any concrete below the horizontal reinforcement at all; even if the concrete is seriously damaged as high up as the neutral axis, the floor can be supported by shores, the damaged concrete removed, and new material substituted for it; the bond between the new and the old material, with the assistance derived from the steel, will be sufficient to hold the new material in place so that it can protect the steel against fire and corrosion, and resist the compressive web stresses, quite as well as in the first instance. In other words, adequate repairs are possible without complete renewal.

To summarize the advantages of attached web members in a fire: The concrete surrounding the lower bars, being free from severe shearing stresses due to the longitudinal stress in the bars, will not fall off so easily; if it does fall, more heat will be required to produce collapse or fatal deflections, and in the absence of these, repairs are possible; whereas, without the attached web members, reconstruction is necessary. Unattached web members, either vertical or inclined, even though they be wrapped around the main bars, will not suffice, for they cannot, in any case, transmit longitudinal stresses into the main bars except through the concrete; if they are vertical, and wrapped around the main bars, this mode of attachment is not sufficient to withstand the diagonal thrust due to compressive web stresses in the concrete. Vertical web members, thoroughly attached to the main bars, without depending on the concrete, would no doubt be sufficient—the composite beam being analyzed, in this case, as a Howe truss; but, with rigid attachment, it is just as easy to use inclined web members, and thus locate them along the action lines of the forces they are intended to resist.

It has been objected, to the use of attached web members, that it involves the use of patented bars. The writer, himself, once thought so, but, after looking into the state of the art, is now convinced that the principle involved is not patentable, but that there are valid patents on ways of applying it; no doubt other patents could be



LONGITUDINAL SECTION THROUGH THE STEEL, AND CROSS-SECTION
AT THE MIDDLE POINT OF
A TYPICAL GIRDER
USED IN THE
WAR COLLEGE BUILDING

Note: Slab Reinforcement not shown on this drawing.

FIG. 5.

secured, on new methods of attaching the web members, should any be developed. At the War College, there being no patented bar on the market in which the spacing of the web members could be varied in accordance with the actual distribution of the stresses, the writer used rectangular bars, with holes drilled from the solid, along the axis, at intervals depending upon the distribution of the tensile web stresses; round bars, cut to a length slightly greater than twice the length of the desired web member, were heated in the middle, thrust through the holes above described, bent up on either side at the proper angle, then hammered down on the flat bars, so as to be upset into the holes, and to grip the flat bars sufficiently hard to enable the assembled reinforcement to be easily handled as a whole. Incidentally, these bars, of open-hearth steel, with all the labor involved, cost, ready to set, nearly $15 per ton less than patented bars which would not have met the requirements as to the distribution of web members. Drawings of types of the War College work are shown in Figs. 5 and 6.

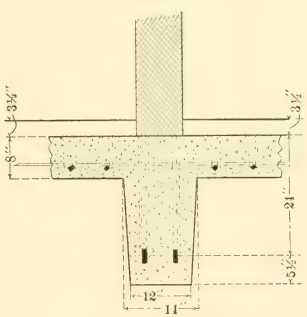
A point ordinarily of vital importance in the practical execution of work is the bond between the floor slab and the part of a beam or girder below it. It almost always happens, as a matter of convenience and economy in execution, that the concrete for that part of the beam below the slab is first deposited, and then allowed to set for quite a while—several hours, or even a whole day—before the concrete for the slab is deposited. This is a matter of secondary importance if properly designed and rigidly attached diagonal web members are used, otherwise it is one of serious importance, and fraught with great danger. There is good reason to believe that many failures in practice have been due to this one condition, and that many structures now standing would fail from the same cause under loads only slightly greater than their working loads, should they ever be so tested.

In this connection, it may also be pointed out that, the use of attached web members, extending to the top of the beam, makes it possible to adopt a concrete for all parts below the neutral axis, for its fire-resisting qualities chiefly, and one for the parts above the neutral axis, for its compressive strength. The two qualities seem not to be combined, as a rule, in the same concrete. Well-made cinder or clinker concrete is much more fire-resisting than stone or

gravel concrete, but it is not nearly as strong. It is more than strong enough, however, to resist all compressive web stresses, and this is the only necessary structural duty of the concrete below the neutral axis, if attached web members are used.

If the writer's deductions, from the results of tests and the theoretical study of the subject, as partly outlined in this paper, are well founded, the following principles may be stated, including some that are the result of executing reinforced concrete work by hired labor, under circumstances demanding the utmost economy.

1.—In designing the section of maximum moment, the maximum economy, in dollars and cents, will result when $A = \frac{b\,d}{p}$; if, by



ENLARGED SECTION THROUGH A GIRDER IN THE WAR COLLEGE BUILDING, SHOWING WEB MEMBERS IN MORE DETAIL, ALSO REINFORCEMENT OF SLAB. THIS GIRDER WAS PUT IN PARALLEL TO THE SLAB REINFORCEMENT TO CARRY A BRICK WALL.

FIG. 6.

Equations 1 to 5, this gives too much steel, then the maximum allowable percentage of Equations 1 to 5 will give the greatest attainable economy, in dollars and cents.

2.—It is not worth while to consider the variations in the applied moment, due to variations in d, unless extreme accuracy is desired, or unless minimum weight is desirable; but an allowance for dead load must be included, in computing the value of M, in any case.

3.—For all girders and beams—and for all flat slabs, if the best results are sought—there must be, in addition to the main horizontal

reinforcement, web members, arranged along the lines of tensile web stress, and rigidly attached to the horizontal reinforcement; they must be spaced so that their aggregate area of cross-section will increase in proportion to the increase in the total web stresses, as the abutments are approached; some web members should be added as counterbraces, according to circumstances.

4.—The adhesion, or bond, of that portion of each web member traversing the compressed part of the concrete, should be sufficient to develop the strength of the member, and the attachment of the member to the horizontal reinforcement should be equally strong. Each horizontal bar should have its own set of web members. Mechanical bond should be used in the upper part of the web members, if there is the least doubt as to the adhesion.

5.—Cheap and easily attached clips, forming small feet, should be used to hold the horizontal bars at the proper distance above the centering; these bars should be of rounded section, as nearly circular as possible, so as not to cause unnecessary shrinkage stresses in the concrete, and so that the wet concrete can be readily made to flow under them. This will give a better result, at less expense, than the current practice of depositing first a thin layer of concrete, and then bedding the steel in it.

6.—As reinforced concrete becomes monolithic in setting, the girders, beams, etc., will act as continuous girders, whether or not that is desired by the designer. This condition might as well be accepted, and reverse reinforcement used at the proper points, to avoid unsightly and often dangerous cracks, if for no other reason. If this is done, it is entirely permissible to reduce the bending moment at the middle section very materially, thus saving some steel in the main reinforcement. The proper distribution of web members in this case requires special attention.

Possibly the writer's conclusions in this paper may not be upheld by future tests, but he thinks they will; practically, no well considered and accurately conducted tests have been made with a special view to determining the points discussed herein. The writer thinks they are at least worthy of experimental determination, and has submitted this paper mainly in the hope of arousing the necessary interest in the subject.

Referring to the paragraph immediately preceding Equations 46 and 47, if the value of a, in terms of the total moment, be substituted in the expression, $4\ a\ d$, for the total volume of the web members, there results, $\frac{4(m + m')}{h\ f_s}$, in which m and m' have the same signification as in Equation 18. As long as $m + m'$ is constant, the total volume of the web members is constant, that is, it is independent of d. On the other hand, under the same assumption, a decreases as d increases. Hence, an ideal system of reinforcement must provide for web members of variable length and spacing, otherwise, when a is properly determined, the web members will often contain too much or too little metal.

It should be pointed out that the expression, $4\ a\ d$, for the volume of the web members, is really based on the assumption that the web stresses have been determined by analogy with a multiple-intersection Pratt truss. If they are analyzed, as indicated in Fig. 4, the amount of tensile web reinforcement will be reduced by just 50%; but, in this case, special pains must be taken to enable the horizontal components of the compressive web stresses to be properly transmitted into the main reinforcement. Under uniform loads, it will be found that the stress in the web member, 5′-3, Fig. 4, for example, will be equal in amount to that in 3-3′, but of opposite sign. The increment in chord stress, from 2-3 to 3-4, however, is equal to the sum of the horizontal components of the web members meeting at 3. Hence, the attachment of 3-3′, alone cannot be depended upon to transmit the total increment of chord stress, but a special bonding device must be provided to take up the horizontal component of 5′-3, or else the adhesion of the main bars must be depended upon, for this purpose. Since the lines of web stress in Fig. 4 are practically those which actually exist in a reinforced concrete beam, and since the compressive web stresses will be parallel to the lines, 5′-3, 4′-2, etc., regardless of the form of tensile web reinforcement, and regardless of whether or not it is attached, it is easy to see why unattached web members are wholly inadequate, no matter how they are arranged. As bearing upon the necessity for attached, diagonal web members, some tests made by the Chicago, Milwaukee and St. Paul Railway, and reported by Mr. J. J. Harding,

show very clearly the value of such web members, even when not arranged strictly as theory demands. An extract from Mr. Harding's paper, describing these tests, was published in *The Engineering Record* of November 11th, 1905. This paper was finished before the writer saw this number of *The Engineering Record*, otherwise the results of the tests would have been referred to before.

DISCUSSION.

Mr. Watson

WILBUR J. WATSON, M. AM. SOC. C. E. (by letter).—Many of the larger cities have building codes which prescribe certain assumptions which must be made in computing reinforced concrete building work, and some of them require that copies of the computations shall be filed with the city authorities. Under such restrictions, the engineer is not at liberty to exercise his own judgment as to the formulas which he shall use, but must select those which will give rational results and still comply with the requirements of the building code, which are often irrational. One of these sets of requirements is as follows:

First, that the ratio of the modulus of elasticity of steel to that of concrete shall be taken as equal to 12;

Second, that the stress in any fiber of concrete shall be assumed to be directly proportional to its distance from the neutral axis;

Third, that the unit stress in concrete, due to bending, shall not exceed 500 lb. per sq. in.;

Fourth, that the unit stress in steel shall not exceed 16 000 lb. per sq. in. under bending loads.

The writer does not wish to be understood as defending the foregoing requirements, especially the first two, which are shown by the author, and also by George H. Blakeley, M. Am. Soc. C. E., in his excellent analysis of the subject published in *The Engineering Record* of May 27th and June 3d, 1905, but is discussing the matter from the viewpoint of the practical designer who is compelled by law to conform to those restrictions. In proportioning the quantity of steel, the writer uses a value of 0.85 of the distance from the top of the beam to the center line of the steel reinforcement as the effective depth of the girder, as recommended by the author. The writer does not like the idea of using multiplied stresses in the analysis of reinforced concrete beams, as it seems to be more rational, and more consistent with the method of analysis used for other types of beams, to use the working stresses assuming the modulus of elasticity of concrete to be constant under the ordinary working stresses. It is often argued, in favor of complicated formulas for designing reinforced concrete beams, that the tests of beams show a greater strength than the more rational and simple formulas would indicate, but such is also the case in regard to the ordinary steel-plate girder. Most writers who deal with this subject place too much reliance on the ultimate strength of a reinforced concrete beam.

It is pretty well demonstrated by tests that, when the stress in the steel rods reaches a comparatively low value, cracks will begin to appear on the tension surface of the concrete. It seems to be the

Mr. Watson.

idea that it is safe to disregard this cracking of the concrete in tension, entirely on the supposition that, provided the stress in the steel does not exceed the yield point or elastic limit of the same, these cracks will close up on the removal of the load, and no harm will be done to the beam. Now, the writer is a skeptic on this point, and, until much more definite data are obtainable, relating to the cracking of such beams on their tension surfaces, and the consequent exposure of the reinforcing rods to corrosive influences, it would seem to be wise for conservative designers to use low unit stresses in the reinforcing steel, in order to reduce, as far as possible, the strain of the concrete in tension.

Granting that it is advisable to use a low unit stress in the steel reinforcing bars, there appears to be very little advantage to be gained in using a high-carbon steel, or, in many cases, a distorted bar. The only advantage that appears to be gained by the use of high-carbon steel is the increase in the factor of safety obtained, provided that such factor of safety be based upon the ultimate failure of the beam. If, however, the factor of safety be based upon the point at which damaging cracks occur in the tensile surface of the beam, the factor will be the same for soft steel as for high steel bars. It will be found that by using a low unit stress in the steel and by using small-sized bars in order to obtain a large adhesive surface, it is often unnecessary to use distorted bars, as conservative values of the adhesion of the concrete to plain bars will suffice to transfer the web stresses to the bars. The superiority of soft or medium steel bars over high carbon bars lies in the greater reliability of the former and the fact that they may be safely bent cold in the field. The writer is firmly of the opinion that when high carbon bars are used, as they very often are, they should never be allowed to be bent at all, as they are very likely to break in the bending, if bent cold, or to be improperly treated, if heated and bent. The writer knows of several cases where high steel bars have repeatedly broken while being bent for use in reinforcing concrete work. If, however, high carbon steel is used, and also high working stresses, then it will often be found necessary to use distorted bars, as in many cases the adhesion of the concrete to the steel will not then suffice to transfer the higher stresses. The writer is not quite convinced of the necessity of attaching web members rigidly to the horizontal bars. The method practiced by the writer has been to use vertical U-bars which are placed in the forms first, the horizontal bars are then placed upon them and are supported by them, the U-bars being hung at their upper ends upon a longitudinal bar supported by blocks placed upon the floor forms. In this way the vertical rods are drawn up tight against the horizontal bars and are thus enabled to transfer tensile stresses directly to the latter. As the U-bars are held by the concrete from slipping along the hori-

zontal bars, the writer cannot see why this is not as good as a rigid attachment to the horizontal bars. The system is very convenient in the field. The writer agrees with the author that web members should be used in nearly all cases of beams, and especially when the beams and girders are placed and allowed to set before the floor slab is laid, which seems to be a common method of procedure. Mr. Watson.

CLARENCE W. NOBLE, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this able paper with a great deal of interest. It is particularly opportune, as the use of reinforced concrete is becoming very general, before many of the theoretical points connected with its design have been definitely decided, and before engineering practice in this regard has been in any degree standardized. For example, there is in common use a bending moment formula which gives an ultimate value for a given beam 100% in excess of the value given by another formula also generally used, and both formulas are sanctioned by practice. The variation in practice regarding the use of shear bars is another case in point. It is to be hoped, therefore, that this paper will be very fully discussed. Mr. Noble.

Some time ago the writer, by graphical methods, reached the conclusion derived analytically by the author, namely, that economical design requires that, unless prevented by other considerations, the steel reinforcement should develop the crushing strength of the concrete when its own elastic limit is reached. Just what percentage of steel is necessary to develop this crushing strength is a question on which opinions differ. A. N. Talbot, M. Am. Soc. C. E., found that in 1-3-6 concrete beams 60 days old the crushing strength of the concrete would not be developed by less than 1½% of mild steel or 1% of high-carbon steel. W. K. Hatt, Assoc M. Am. Soc. C. E., found that 2½% of mild steel did not develop the crushing strength of a 1-2-4 rock concrete beam loaded centrally. Edgar Marburg, M. Am. Soc. C. E., found that a beam reinforced with 1.19% of Ransome bars having an elastic limit of 58 000 lb. per sq. in. did not fail by the crushing of the concrete. These tests show allowable percentages of steel considerably in excess of those derived analytically by Captain Sewell. On the other hand, A. L. Johnson, M. Am. Soc. C. E., working analytically, reached a result slightly below those under discussion. Theory and practice here seem to part company. As the maximum of economy is attained in beams highly reinforced, this is a point concerning which investigation can be profitably continued.

Economy in the choice of materials is a matter which is pertinent to the subject under discussion. In designing roofs, for example, it is frequently found theoretically possible to use slabs much thinner than the 3-in. limit fixed by good practice. In such cases, using the 3-in. slab, if the reinforcement used is less than 0.5 of 1% of high-carbon steel it is advisable to use cinder concrete, as the

Mr. Noble. material is cheaper and affords sufficient strength. The choice between the various forms of patented and unpatented reinforcing bars also resolves itself into a question of how much total elastic limit in a satisfactory form can be bought for a cent. If a given amount of tensile strength, together with the necessary bond between the concrete and steel and sufficient provision for shearing stress, can be supplied by plain bars at less expense than by patented bars, then, obviously, the cheaper bars should be used. This brings up a discussion of the nature of the bond between steel and concrete.

The most extensive series of tests of the union between steel bars and concrete, known to the writer, was made at the Massachusetts Institute of Technology.* The report of these tests has also been widely circulated by the patentees of a certain kind of deformed bar. A number of plain and deformed bars were embedded for various lengths in 1-3-6 concrete blocks. These bars were pulled out by direct tension from an Olsen testing machine. The bars extended entirely through the concrete, and observations were taken on the free end to determine the first slip. The tests on round and square plain bars of structural steel are shown in Table 1.

TABLE 1.—Tests of Adhesion Between Plain Bars and Concrete.

Size and kind of bar.	Depth embedded, in inches.	Adhesion, in pounds per square inch exposed to concrete.	Stress per square inch in bar.
¾-inch, round	24	271	38 400
¾-inch, square	24	274	35 200
¾-inch, round	31	255	42 200
¾-inch, square	31	243	40 400
¾-inch, round	36	219	42 200
¾-inch, square	36	221	42 700

It will be noted, from Table 1, that in every case the bars slipped in the concrete at a point where the steel was strained to more than its elastic limit. This makes an obviously unfair test, for, when the bar begins to elongate, the entire load is taken off by the concrete immediately around the point where the bar enters it, and consequently the adhesion between the concrete and that portion of the bar farther within the block will not be stressed. The resultant action would be similar to that in tearing a piece of paper, which can withstand a considerable tensile stress applied uniformly over the sheet, but fails at once when the load is concentrated at one edge. Such an action shows very plainly in the tests given in Table 1, as the longer bars failed at considerably lower average adhesive stress per square inch than the shorter bars. This condition

* *The Railroad Gazette*, September 18th, 1903.

probably exists to an extent in tests made within the elastic limit of the steel, although it would never occur in a reinforced concrete beam under actual working conditions, as there the stress is applied gradually by the concrete acting on the bar, and the ends of the bar are unstressed at all times. Overlooking this fact, however, Table 1 would show that one can expect an adhesion between concrete and plain steel bars of more than 275 lb. per sq. in. Professor Hatt finds values varying from 636 to 756 lb. per sq. in., and states that, after the rod starts, from 50 to 70% of its original adhesion remains, due to the grip of the concrete. Bauschinger obtains values ranging from 570 to 640 lb. per sq. in. If experimental tests are of any value, an assumption of 250 lb. per sq. in. for adhesion is certainly conservative. This being the case, the statement made by the author that the sum of the horizontal components of the stresses in the web members on each side of the center of the beam should be sufficient to develop the strength of the flange reinforcement, can only be justified as economical practice on the ground that the bond between the steel and the concrete below the neutral axis of the beam is unreliable. He must also consider the ability of the concrete to resist shear as worthless. The majority of engineers dealing with reinforced concrete will probably not agree with the author on these points. Mr. Noble.

J. W. Schaub, M. Am. Soc. C. E., talking before the students of the Armour Institute of Technology, compared experiments with rods embedded in concrete with those made with concrete pats allowed to set upon steel plates. He concluded that the adhesion between concrete and an embedded rod is due to two causes. The first is the formation of a slightly soluble silicate of iron having a cementitious value in tension of 22 lb. per sq. in. The second is the gripping effect of the concrete due to shrinkage during setting, which pushes particles of cement and sand into the minute unevennesses of the surface of the bar. The silicate of iron causes that portion of the adhesion found by Professor Hatt to be lost with the initial slip. Assuming either Hatt's or Bauschinger's values as correct, all the silicate can be dissolved out of the concrete, or can attach itself to mill scale instead of to the steel itself without reducing the adhesion to 250 lb. per sq. in. Evidently, therefore, this can be assumed as a safe ultimate value.

The advocates of deformed bars advance the argument that the lengthening of a bar under tension tends to decrease the diameter of the bar and thus relieve the grip of the concrete. Now, it is not at all certain that the lengthening of the bar within the elastic limit will cause a corresponding loss of diameter. Certainly, the tendency is to separate the molecules of steel forming the bar, and it is hard to see how this tendency can be transmitted from molecule to molecule along the length of the bar unless some separation

Mr. Noble. actually takes place. But, assuming that the diameter is reduced correspondingly, and that a high-carbon bar having an elastic limit of 50 000 lb. per sq. in. is stressed from zero up to that limit, it would only be increased by $\frac{5}{2900}$ of its original length. A corresponding reduction of area would mean that the reduced radius would be 0.99914 of its original length, thus reducing the radius of a ½-in. bar by 0.000215 in. and the radius of a 1-in. bar by 0.000430 in. If it be assumed that the concrete grips the bar with a pressure of only 250 lb. per sq. in., and that the modulus of elasticity of concrete is 2 400 000, it would only relieve the compression for a distance of 2 in. from the ½-in. bar if the concrete should follow the receding steel for the entire distance of 0.000215 in. Evidently, there is no danger that the reduction in diameter of the bar will cause the particles of the concrete to withdraw from the interstices of the steel surface.

A refusal to recognize the shearing value of concrete can only arise from the belief that when the beam has taken an appreciable deflection, so that the concrete below the neutral axis has been separated by minute cracks, this shearing value disappears. This is doubtless the case where such cracks occur, but, taking place as they do at points where the steel is stressed the highest, they occur at points of lowest shear, and, even here, unless the steel is loaded to more than its elastic limit, do not go above the neutral axis of the beam. Even if the beam is designed for a concentrated moving load, the concrete above the neutral axis near the middle of the span would be as well able to take the maximum shear coming on it as would the concrete in the full depth of the beam near the ends. Therefore, it would seem to be only necessary to provide vertical reinforcement for shear in excess of that which can safely be carried by the concrete.

It is the writer's belief that economical designs are best obtained by the use of a comparatively large number of small undeformed bars for horizontal reinforcement. This insures ample bond between the steel and the concrete for all but very short and deep beams. As the ends of the beam are approached, the flange stresses become of relatively less importance, and these bars are then, one or two at a time, turned up at an angle of from 30 to 45° into the concrete and continued along the top of the beam to the point of support, where they bond into the wall or adjacent beam. They thus serve successively as positive-moment bars, shear bars, and negative-moment bars, meeting in each situation the greatest need of the beam, and doing it always with a maximum of economy.

Mr. Kreuger. I. KREUGER, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Sewell gives some very valuable suggestions for the design of reinforced concrete beams. The writer believes, however, that until more thorough knowledge of the properties of concrete is obtained, the

straight-line formula is the most satisfactory one for computing reinforced concrete girders. Mr. Kreuger.

There is hardly a prominent engineer in Europe or America dealing with concrete who has not established a formula of his own, usually built upon an assumed variation in the stress-deformation curve of the concrete. The factors which influence this curve seem to be so many and of such a complex nature as to make hopeless the task of obtaining a result, true for all classes of concrete. While numerous observations as to the deformations of concrete under compression have been made in Europe and America, the widely different results obtained tend to make them rather confusing, and, under these conditions, the author's stress-strain curve seems to be an unwarranted refinement.

The simplicity of the straight-line formula, and the fact that it is accepted for the building laws of many prominent cities, are factors greatly in its favor. It seems to the writer, however, to be of secondary importance to establish the true stress-strain curve of the concrete in compression, as all tests show that this curve is quite different from the stress-strain curve of the concrete subjected to bending. This is not always clearly recognized, but the customary provision to calculate a higher value for bending compression than for direct compression is an admission of, and an allowance for, the errors of the present method of computing reinforced concrete girders.

Such evidence can also be found in the tests mentioned on page 279 where the steel was pulled apart, though the percentage of steel was more than twice the amount recommended in the table on page 261.

If one insists upon computing concrete girders in a manner similar to that used for steel beams, and if one maintains the assumption that the tensile strength of concrete should be neglected, the results of the bending tests executed on reinforced concrete girders would lead to the assumption of a stress-strain curve following closely a rectangle. The diagram for such a curve would then have the appearance of Fig 7.

The writer, however, believes that, until more thorough tests have been made on this subject, the straight-line formula at present in use is the most convenient way for calculating reinforced concrete beams.

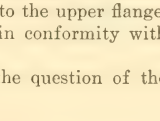


Fig. 7.

The conclusions to which the author comes, that for economical design as large a percentage of steel as the concrete will stand should be used, and that no steel should be put into the upper flange, seem to be thoroughly sound, and are probably in conformity with the experience of most designers.

Regarding T-beams, the author brings up the question of the

Mr. Krueger. flange width which should be counted on. The praxis on this point varies greatly. The building laws of several cities, notably Cleveland and Buffalo, allow a width of ten times the width of the beam. It is contrary to all authorities on this subject, however, to use such a large flange width, and the figure given by the author—three times the width of the beam—seems to be much more satisfactory. It is difficult, however, to see in what way the allowable flange width is influenced by the width of the beam, and it would seem to be more correct to make it dependent upon the length of the span. For certain cases where there is danger of buckling, the thickness of the slab may be the governing factor. It has been the writer's praxis to allow a total flange width of one-sixth of the length of the beam.

The system of web reinforcement described by the author is a very interesting feature of the paper. It may be said that the value of web reinforcement has been clearly established, and most engineers agree upon this point.

Of the author's final remarks, the writer considers No. 6 particularly worthy of attention. If concrete girders were generally calculated as continuous, and the steel reinforcement arranged accordingly, a great saving could be made, and at the same time there would be a gain in safety.

Mr. Dana. RICHARD T. DANA, ASSOC. M. AM. SOC. C. E. (by letter).—The author has taken the longest step, so far, toward placing the design of concrete structures on a strictly scientific basis. The mathematical work is admirable, and the deductions, in the main, seem to be accurate. The writer does not entirely agree with the author in placing the cost of steel at 3 cents per lb., as it would seem that $2\frac{1}{2}$ cents would be a closer figure, bringing the average value of p to 60. The author has designed his formula for ultimate strength. The writer is of the opinion that it is better practice to design for working stresses, for the following reasons:

1.—A very much smaller part of the stress-strain curve is brought into play, and to the stress-strain curve a close approximation may be made by a straight line, thus permitting the use of a straight-line formula.

2.—The factor of safety for concrete should, at times, be decidedly different from the factor of safety for steel in tension. If a beam is designed on the basis of the point of failure of the steel and concrete, or on the basis of the point of failure of the steel, namely, the elastic limit of the steel, and a percentage, say, 80, of the point of failure of the concrete when the beam is under vibratory loads, the concrete will be working under a disadvantage, as compared with the steel, and therefore the concrete will be overworked, which would result ultimately in weakening the beam. It would

Mr. Dana. seem to the writer, therefore, to be better practice to design for the actual safe stresses of the two materials.

The formula worked out by the author is exceedingly ingenious, and is the simplest one known to the writer, having regard to rigidity of reasoning. In beam formulas, however, the writer prefers to use the method delevoped in the last few years in France by Coignet, Tedesco, and Maurel. The method replaces the steel in a section with a quantity of concrete equivalent to the area of the steel multiplied by $\frac{E_s}{E_c}$. The section, thus reduced to a homogeneous one, is referred to a neutral axis passing through the center of gravity, and the moment of inertia of the entire section is obtained on the basis of the concrete alone. This method results in extremely simple formulas for rectangular T-beams, and is applicable to the rapid analysis of very complicated shapes. It is the only rapid method known to the writer whereby a hollow section of irregular area, with various arrangements of reinforcement, can be calculated accurately. The value of N, or the ratio between the coefficients of elasticity of steel and concrete, will, of course, vary for different conditions, and it seems to the writer to be essential, in any general formula, that N be chosen by the designer for the special conditions of practice, and that it should be in shape to insert readily in the formula.

Concerning the question of web stresses, in the writer's opinion, the concrete beam is analogous, not to a Pratt truss, or to a Warren girder, but rather to a Howe truss. While the method of reinforcement given by the author is unquestionably excellent, and will produce a safe beam, it is believed by the writer that an equally safe and rather cheaper beam can be made with vertical stirrups spaced at distances equivalent to the depth between the centers of tension and compression, with special attention paid to what in a plate girder would be end stiffeners.

With the practical reasons, given by the author on pages 281 and 282 for the use of stirrups or web members, the writer fully concurs, except in the argument based on the reconstruction of such beams in place. Where the heat from the fire has been so great as to dehydrate the cement, the steel is likely to have suffered greatly, and even if the concrete were replaced, the beam would probably not have anything like its original strength, particularly if cold-drawn wire or cold-twisted steel were used for the reinforcement.

The thanks of the engineering profession, as well as of this Society, are due to the author for this admirable paper.

Mr. Turner. C. A. P. Turner, M. Am. Soc. C. E. (by letter).—While the writer has designed a great many buildings of reinforced concrete, and while, in every case where his instructions have been carried out, the resulting work has been stronger than represented by him,

Mr. Turner. he will frankly state that though he has been able to design safe and satisfactory work at a low cost for construction, he regards it as impossible in the present embryonic state of knowledge of the properties of reinforced concrete to attempt successfully anything approaching a valuable general mathematical discussion and investigation of the economical design of reinforced concrete floor systems. This statement is made in fairness to Mr. Sewell in order that such criticism as the writer offers may be better understood, and that such theories as are presented with the criticism will be accepted in their true light, as a merely suggested explanation of observed facts.

It happens too often in a mathematical discussion that the theorist starts out with an assumed premise, then proceeds to build an elaborate mathematical theory thereon, and forgets in his summing up that his reasoning, however accurate his mathematics, is, after all, based on the assumed premise rather than on fact. This seems to the writer to be the basis of the author's expressed belief that, in his short discussion, he has brought out the real principles of economic design.

The author's assumptions will be taken up in detail, in order to arrive at the value of his conclusions. First, he asserts that no extensive system of concrete floors can be economically designed without the use of rolled-steel beams or concrete ribs. Now, the writer's experience, in paying the cost of this work in labor and materials, is that where the panels are not greater than 25 ft. square, for a guaranteed test load of 200 or 300 lb. per sq. ft. over the full area, a plain slab without ribs costs less than one with ribs. In warehouse work, it is perfectly feasible to put up a building with columns at 16-ft. centers, with a floor of 7½-in. rough slabs, using no ribs at all, and test it with 800 lb. per sq. ft. without injury to the construction. Furthermore, it can be put up at less cost without the ribs, and will require less metal, as the load will travel more directly to the supports, instead of around a corner, as in the case where beams are used. This method of construction is outlined in Figs. 8 and 9.

Now, floor slabs are made anywhere from 5 to 25 ft. in span, and, on this basis, can a discussion, of the economic relations of the slab and ribs, which starts out by assuming a constant value for the thickness of the slab be regarded logically as of any value whatever? Aside from this little oversight, the author fails to realize that if beams or ribs are at 5-ft. centers, the rib centering costs just four times as much as if they were at 20-ft. centers. This, also, is an item which is not fixed by the problem in most cases, but by questions of economy only, and must appear in the discussion, if it be of any practical value whatever. Hence, the cost of centering, which the author assumes as a constant, is in reality a very variable one. Again, the

floor system is but a part of the whole, and, as the same load can be carried to the footing more cheaply through one column than through two, this item enters into the discussion, if it be complete. Mr. Turner.

Now, in any physical research, a mathematical theory is of value only as it agrees with and explains the results of practical experiment, and, if the author's theory is tenable, as a general basis of economic design, it should fairly explain the results and facts developed in the course of practical work. Take, for example, the

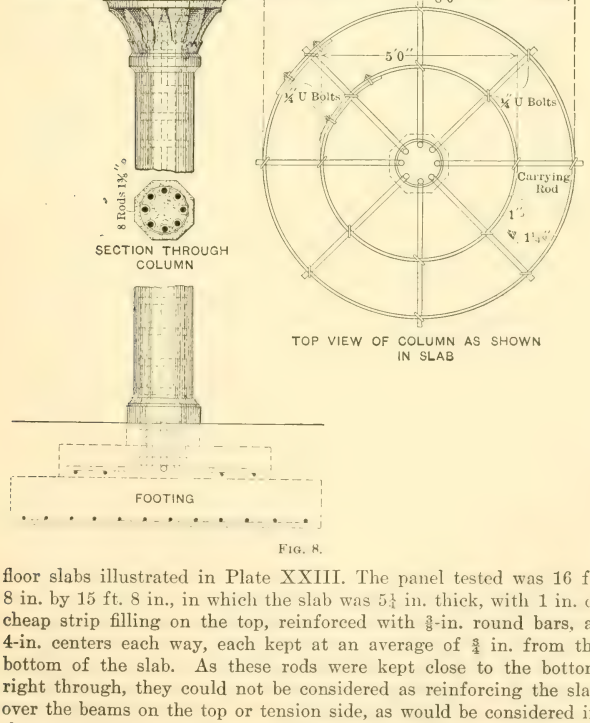


FIG. 8.

floor slabs illustrated in Plate XXIII. The panel tested was 16 ft. 8 in. by 15 ft. 8 in., in which the slab was $5\frac{1}{2}$ in. thick, with 1 in. of cheap strip filling on the top, reinforced with $\frac{3}{8}$-in. round bars, at 4-in. centers each way, each kept at an average of $\frac{3}{4}$ in. from the bottom of the slab. As these rods were kept close to the bottom right through, they could not be considered as reinforcing the slab over the beams on the top or tension side, as would be considered in the ordinarily accepted theory. The rods were long enough to go

Mr. Turner. just over the top of the beams and were hooked at the end. A test load of cement in sacks of approximately 900 lb. per sq. ft. was applied to the slab and kept inside the beam lines so as to give a straight shear load on the slab. The strip filling was a very weak mixture of lime, cement, and sand, so that it could have added little to the strength. Disregarding the strip filling and calculating the moment at $\frac{1}{8}$ W L, or, what is probably more nearly correct, $\frac{1}{10}$ W L, in the present case, distributed equally between the two systems of

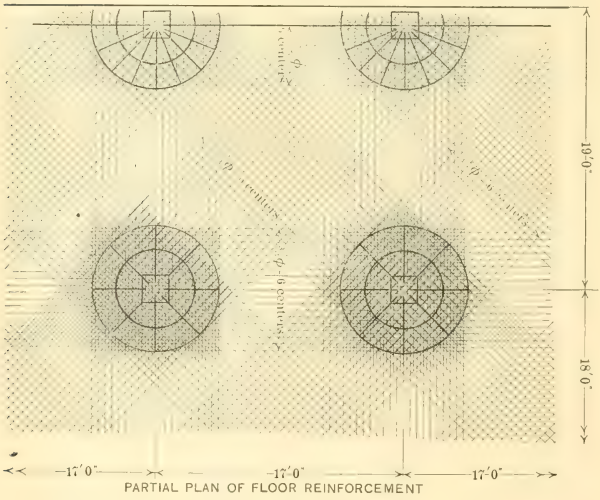


FIG. 9.

rods, gives, in the author's formula: $M = h\,d\,a\,b\,t_s = 0.846 \times 4\frac{1}{2}$ in. $\times \frac{0.333 \text{ in.}}{12} \times 12$ in. $\times$ 40 000 = 50 760 in-lb. for the ultimate strength. Now, the moment of the load, $M = \frac{1}{10}$ (450×15) $16 \times 12 = 129\,600$ in-lb.; in other words, the moment of the applied load is 2.6 times the yield point value of the steel, as indicated by the author's formula, leaving the dead weight to care for itself, or be carried by the cement. The actual deflection of the slab, under the test load at the center, was only $\frac{5}{32}$ in., and it would require a deflection of $1\frac{1}{2}$ in., at least, even to crack such a slab. From this

FIG. 1.—WAREHOUSE, N. W. KNITTING CO., SHOWING CONSTRUCTION WITH 5¼-IN. SLABS, 16 FT. 8 IN. BY 15 FT. 8 IN.

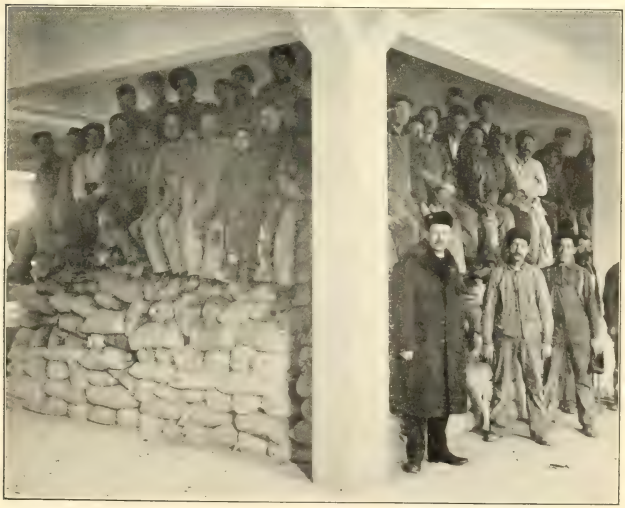


FIG. 2.—WAREHOUSE, N. W. KNITTING CO. TEST LOAD OF 900 LB. PER SQ. FT. ON 5¼-IN. SLAB, 16 FT. 8 IN. BY 15 FT. 8 IN.

fact, it is a fair inference that the strength of the construction was at least from 700 to 800% of what might be expected from the author's economic theory; and either excessively strong concrete is being made or the author's theory is very weak indeed. It should be noted that the concrete tested was only about 7 weeks old, and, probably had not developed more than 70% of its ultimate strength, due to the slow drying at that season of the year; further, that the deflection of the beams was $\frac{1}{16}$ in., leaving $\frac{3}{32}$ in. deflection of the slab between the beams. Mr. Turner.

Mr. Marsh, in his work on reinforced concrete, Part V, writes as follows:

"It may be that we are wrong from the commencement in attempting to treat it (reinforced concrete) after the manner of structural ironwork. * * * The molecular theory, *i. e.*, the prevention of molecular deformation by supplying resistances of the reverse kind to stresses on small particles, may prove to be the true method of treatment for a composite material such as concrete and metal."

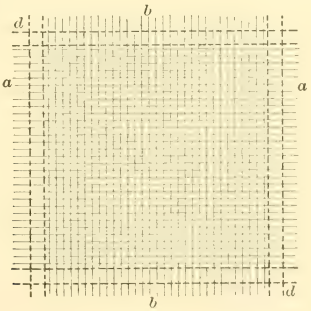


FIG. 10.

It seems to the writer that the difficulty with the author's theory is in the fact that he is endeavoring to treat the dual material, as Mr. Marsh says, after the manner of structural ironwork. He practically starts out with the assumption that the concrete can be economically reinforced in one direction only, or assumes, following certain other writers, that where the reinforcement is applied in two or more directions, the same treatment holds; and that the stress has only to be divided between the various systems, and that otherwise, their joint action can be legitimately ignored.

Glancing now at Fig. 10, the following facts appear in evidence under any bending of the slab: The rods, *aa* and *bb*, the reinforcement, are in tension and along the diagonal lines; *dd*, the lower

Mr. Turner. fibers, are also in tension. Now, these forces must be balanced by compression in the upper part of the slab. Where the reinforcement runs in one direction only, this stress is cumulative toward the center, but where it is in two or more directions it may seemingly be carried largely by lateral arching. See Fig. 11.

Again, there is little exact knowledge as to how great an increase in strength may be obtained under the conditions that one compressive stress tends to balance the deformation of another. M. Considère's researches have thrown some light on this, and are, perhaps, an indication as to what may be accomplished along analogous lines. It may well be that the expert physicist may be able to devise an apparatus to measure the molecular stresses by thermo-electric means, as the writer has succeeded in doing in steel. For such investigation, a concrete of sand and cement only might simplify the problem.

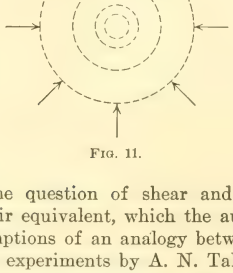


FIG. 11.

Next consider the question of shear and bars with attached web members, or their equivalent, which the author seems to fancy because of his assumptions of an analogy between a concrete beam and a truss. In the experiments by A. N. Talbot, M. Am. Soc. C. E., these bars indicated a strength of only 87% of that of plain bars of equivalent section, and, in view of their irregular shape, rough-sheared and nicked section, it is surprising that they did even as well as that. That good results have certainly been obtained with them, cannot be disputed, nevertheless, 60% of the strength that may be obtained with plain round bars and good work will pass muster anywhere.

As regards the use of shear members, the writer's experience would indicate that better results may be obtained without them, by simply tying in the skin of the beam or rib with a net. The idea of having the flange reinforcement all in the bottom of the beam, except at the center, is certainly rarely followed by those familiar with practical work, and the author's remarks, based on the assumption that the reverse is true, and his conclusions regarding

FIG. 1.—WAREHOUSE, MINNEAPOLIS PAPER CO. TEST LOAD OF 110 TONS ON 6½-IN. SLAB, 15 FT. 4 IN. BY 21 FT. 6 IN., CENTER TO CENTER OF BEAMS.

FIG. 2.—WAREHOUSE, MINNEAPOLIS PAPER CO. TEST LOAD OF 160 TONS ON 860 SQ. FT. COLUMN SPACING 15 FT. 4 IN. AND 21 FT. 6 IN., LONGITUDINALLY AND TRANSVERSELY, CENTER TO CENTER.

the superior advantages of the attached web member bar, from the fire-proof standpoint, are not substantiated. Mr. Turner.

Comparing the construction by the manufacturers of the attached web member bar with construction where plain bars are used, and referring to the warehouse of the Farwell, Ozmun, Kirk Company at St. Paul as a typical example of the former type, and the warehouse of the Minneapolis Paper Company as an example of the latter, the following tests offer a practical method of judging these claims. In the warehouse of the Farwell, Ozmun, Kirk Company, for the panel tested, the columns were at 13-ft. centers along the main girders, and at 16-ft. centers along the carrying beams, which were spaced at 5.5 to 6.5-ft. centers, and the slabs were about 7 in. thick. The test load was 78 tons of pig iron over one beam, which caused a deflection of $\frac{1}{64}$ in. In the warehouse of the Minneapolis Paper Company the panels were 15 ft. 4 in. by 21 ft. 6 in.; the slabs were $6\frac{1}{2}$ in. thick, over the full panel, with 1 in. of strip filling made of weak mortar; the reinforcement was $\frac{3}{8}$-in. round rods, spaced at an average of 6 in. from center to center each way; the beams ran from column to column in each direction only; the test load was 110 tons (shear load) on the slab, and, later, 50 tons were added to test a 12 by 16-in. beam with 21 ft. 6 in. span. The slab reaction on the beam would be approximately 70 tons. The deflection of the beam was practically inappreciable. Now, if, as above noted, $\frac{3}{8}$-in. rods with an average centering of 6 in. can carry on a $6\frac{1}{2}$-in. slab 110 tons to the beams, at 15 ft. 4-in. and 21 ft. 6-in. centers, the floor slab in the St. Paul warehouse, if the attached web member bar is any good, ought to distribute the load over at least three beams, so that this test was a little greater than the load the floor was designed to carry—500 lb. per sq. ft.—while, in the case of the plain bar design, there was double the load, and on a span one-third longer than that of the building of the Farwell, Ozmun, Kirk Company. There seems to have been in the slab approximately an equal weight of metal reinforcement per square foot in each case, and also per linear foot in the beam, though, in the plain rod design, the clear span of the slab was nearly three times as great, while the beam carried twice the load and was one-third longer in span. Upon such a showing, the attached web member bar would be entitled to rather a scant consideration, if this illustration is a fair one. Figs. 1 and 2, Plate XXIV, show some tests at the warehouse of the Minneapolis Paper Company.

The basis of the author's belief in the web member bar or its equivalent will now be examined critically. He states that he analyzed the web stresses by analogy with a Pratt truss, and, later, at the War College, further study made it apparent that the double-intersection Warren girder was a better analogy. Admitting, for the sake of argument, that it is permissible to draw valuable conclusions

Mr. Turner. from truss construction relative to reinforced concrete beams, according to the author's belief, the Pratt truss was a good analogy and the double Warren girder a better one, but, if an opinion is to be formed in this questionable way, engineers should obtain the best possible analogy. Taking the author's example, the case of the uniform load, and considering an analogy with an inverted parabolic bowstring or deck truss, the reinforcement is of constant section, as recommended by him, the web stresses are all compressive, and the curved reinforcement seems to be ideal in this respect, doing away with his multiplicity of relatively small and bothersome web members, effecting thereby a theoretical economy of 30% of the metal and 75% of the trouble in handling and placing it. This analogical theory, it will be noted, is for uniform loads. There remains, therefore, to treat any combination of live loads in a similar manner for the simple beam. Still proceeding on the author's recommendation of a constant tension chord section, as the compression chord of concrete is constant, one should seemingly look for a suitable analogy in that type of truss which economically discards diagonal panel web members, while complying with the

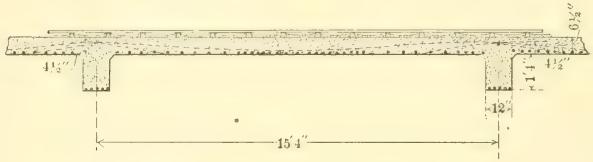


FIG. 12.

above fixed conditions, and this type is evidently represented in the Bollman truss. While, in steel construction, the conditions noted bar it from economical use, with these fixed, it possesses quite a marked advantage over reinforcement planned on the Warren girder order, combined with a dissemination of steel through the concrete and ease in placing it that should be apparent to those having practical experience in this line.

An example of the failure of the theory as applied to a slab which is nearly square has been given, and now the author's formula will be applied to the rectangular slab tested at the warehouse of the Minneapolis Paper Company, 21 ft. 6 in. by 15 ft. 4 in., from center to center, beams 12 in. wide, load 860 lb. per sq. ft., distributed over nearly 13 by 20 ft. in the center of the slab, as shown in Fig. 12.

Weight of construction.....	90	lb. per sq. ft.
Live load.....	860	" " " "
Total	950	" " " "

Mr. Turner.

Using Mr. Dunn's well-known formula for distribution over rectangular slabs, with continuous reinforcement:

$$M_c = \text{Moment over support} = \frac{W\,B}{12} \times \frac{L^4}{L^4 + B^4},$$

B being breadth, and L length of slab. Therefore:

$$M_c = \frac{12\,350}{12} \times 15 \times 0.8 = 12\,000 \text{ ft-lb.} = 144\,900 \text{ in-lb.};$$

$d = 4\frac{1}{2}$ in.; $h = 0.846$; $t_s = 40\,000$.

$M = h\,d\,a\,b\,t_s = 0.846 \times 4.5\ (0.44) \times 40\,000 = 67\,000$ in-lb. = the calculated ultimate strength, while we have tested it to 2.2 times this amount with a deflection of $\frac{1}{1000}$ of the span, and could certainly apply more than 3 times this load before final failure. It should be noted that the value of a used is the average section, the bars being spaced closer together at the center of the slab, in this particular design, and is taken as double for the lap of the rods in the adjacent panel. One would suppose that this arrangement would show greater stiffness than that first outlined, but a test does not seem to bear out the inference, or, at any rate, to give it a positive standing.

Next, consider the question of the character of the test. Was the loading such that the conclusions are warranted? Was there any considerable arching, and to what extent could the surrounding construction distribute the load? When the ratio of the depth of the slab to its span is only 1 to 8 or 10, as in the building of the Farwell, Ozmun, Kirk Company, the slab can distribute a very material part of the load over adjacent beams, but where this ratio increases from 1 to 8 or 10 to 1 to 25 or 35, the stiffness being approximately as the cube of the span, this amount would be but $\frac{1}{15}$ to $\frac{1}{25}$ of the former, where the ratio is only 1 to 8 or 10, perhaps from $2\frac{1}{2}$ to 4%, calculating this action in the former case, the St. Paul test, as great as 60 per cent.

Now, as to the question of the arching of the load from beam to beam, and reducing the load carried by the slab, it may be stated that the sacks were piled with especial reference to reducing this action to a minimum. With a deflection so small, this action, in any case, would be slight, perhaps reducing the calculated moment less than from 5 to 8%, which might be estimated for a sand or grain pile of these dimensions. It may be fairly said of the load that it is similar to loads that would be applied in use in the building, and that the slab load was piled inside the beam lines to give a straight shear load on the slab. Again, tests have been made on slabs with white lead in kegs, piled so that there was no arching, and the calculated strength, by the formula noted, did not agree more closely with the practical results than in the foregoing instances.

Referring now to the author's conclusions as to the economic

Mr. Turner

relation of the cost of the concrete and steel, and, taking in this case a plate girder for analogy, it is considered conservative to disregard the web resistance to bending in a plate girder, just as in the concrete beam the web resistance of the concrete below the neutral line is disregarded. Now, according to general practice, it is calculated that the compression flange should equal the tension flange in cost, that the section of the flanges and consequent cost decreases with the increase in depth, and, by principles of maxima and minima, one ordinarily tries to prove the total cost a minimum when the costs of the web and flanges are equal.

Now, vary the problem by making the tension flange of a special grade of steel and the web and compression flange of common steel, as before, and write the moment equation thus:

$$M = h\ d\ a\ b\ t \ldots\ldots\ldots\

Mr. Turner.

While, from the engineer's standpoint, the writer can see but little opportunity for the use of this economic theory, from the standpoint of the vendor of reinforcement, if he can but convince the purchaser of its accuracy, the commercial possibilities in this line would seem to be very attractive. This commendable feature of the theory, combined with its apparent plausibility, would readily deceive anyone failing to note the confusion of constants and variables involved in the assumptions on which it is based.

The writer will now examine the assumptions under which the value of h was determined a constant, and see how the author proceeds to fill these conditions in his economic theory. These conditions are that the steel and concrete are each worked to a definite and constant allowable limit. Now, in his economic theory, he proposes to vary d and a to balance a constant moment, M, but, arbitrarily, to keep h and b constants. The writer will start with a depth less than the economic depth and double it, and note at these extremes, under the author's assumptions, the variation of the working stress of the concrete. For the shallow beam, the concrete can be worked, of course, only to its safe working stress. When d is doubled, the total compressive stress on the compression chord (the concrete) is cut in two, b and h remaining constant; by the author's economic theory, the area carrying this stress is doubled, making the working unit stress for the concrete, in the second case, only one-fourth of what it was in the first; in other words, according to the author's assumptions, he compares the economic relations of the steel and the concrete on the basis of a rational and fixed working stress for the steel and, as it happens, an actual variable (within the narrow limits of 300 or 400%) working stress for the concrete. This appears to be rather severe on the concrete, as indicated by the practical example of the slab. Upon such a basis of computation, it is hardly surprising that the author concludes that his theoretical economy, based on relative costs, is not attainable. He also notes that his discussion of minimum cost does not contain the ratio between the allowable maximum stresses in the two materials, but fails to note that this ratio, which should be fixed in a rational discussion, is an extremely variable one as involved in the equations from which he essays to draw his conclusions.

The writer, in his remarks, thus far, has not questioned the author's assumption of a constant section of reinforcement, from the economic standpoint. Now, for a simple beam, where the moment varies from zero at the end to $\frac{1}{8}\ W\ L$ at the center, the need of the maximum section for the full length is not apparent. In the Moulton Jordon garage there were some short-span girders of reinforced concrete, 42 ft. from out to out, in which the writer considered that there was a very decided economy in designing them with a variable

Mr. Turner. flange, as he would have done if they had been steel. This saving, of course, would increase largely with the increase in span.

In ordinary floor systems, however, there is the general problem of a series of spans, and the question of the relative economy of continuous *versus* simple beams at once arises. As it is impracticable to vary the section of concrete along the length of the beam, and as the maximum moment for the continuous beam for a uniform load is approximately only two-thirds of that for the simple beam (calculating for an intermediate span), the continuous construction should result in a material saving in concrete.

Considering now the steel reinforcement: The moment at the support is double that at the center of the span, and of the opposite sign. Now, the rods are rolled of constant section, and, if they are carried from the tension flange at the center to the tension flange at the support and lapped into the next beam, there would be required, approximately, one-third the section for half the length and two-thirds the section for the remainder that would be required for the simple beam construction, or half the metal required in the simple beam with constant section of reinforcement. With this arrangement, there is the condition that the maximum flange reinforcement is required at the point of maximum shear, that the moment decreases rapidly from the support toward the point of contra-flexure, allowing this main reinforcement to drop sharply downward without weakening the construction, thus placing the main section of metal in a position to carry the entire shearing strain, without counting on the concrete or depending on the questionable amount of adhesion that may be obtained between the concrete and small and stubby web members.

This simple bend in the bars gives them an anchorage in the concrete which, from the writer's experience, appears to discount any form of nicked-section mechanical bond yet invented.

Owing to the fact that the moment at the center is only half that at the support, the question of reducing the concrete further by reinforcement of both flanges at the support, for part of the length only, should be considered, if the economic theory be complete. This method is followed by the writer.

Bad work, in one instance, executed by an incompetent contractor, on a footing, gave the writer an opportunity of judging the amount of distortion a connection of this character would stand, and he was not a little surprised to be forced to conclude that it could stand, if anything, as great an amount of distortion, without material injury, as could be expected from a structural steel frame with standard riveted connections of the web of the beams to the columns. Such reinforcement is more satisfactory from the standpoint of resistance to lateral or vibratory forces.

Mr. Turner. The fewer joints there are in the concrete, the more uniform it is in strength; and any method of placing it piecemeal, as observed by the author to be done frequently, cannot be too strongly condemned. The cement is that part of the composite material which gives it its strength, and, to the largest extent, its fire-proof properties, and anyone who possesses the temerity to follow the author's suggestion of using a cheap concrete for the lower half of a beam, whether with the attached web member bar or any other, is, in the writer's judgment, industriously looking for trouble rather than economy.

In conclusion, it is safe to assert that no one has a higher respect for true theoretical economy than the busy engineer of construction. This brand of theoretical economy is attainable, and is based on a complete and accurate statement of all the facts entering into the problem. That brand which is not attainable, he immediately concludes, is based either on an incomplete and defective statement of the conditions of the problem, or on inadmissible assumptions regarding them.

Mr. Jonson. Ernst F. Jonson, Assoc. M. Am. Soc. C. E. (by letter).—There is one point in Mr. Sewell's paper to which the writer begs to call attention as not being quite correct.

The author takes the depth of the axis of the horizontal reinforcement below the top of the beam as a basis for his shear computation, instead of the depth of that axis below the resultant or centroid of the compressive forces.

The following proposition is true of all beams:

The total shear on any one cross-section of a beam is equal to the average unit shear at the neutral axis, multiplied by the width of the beam at the neutral axis, multiplied by the distance between the resultant of the compressive forces due to the bending moment and that of the tensile ones.

$$S = s\ b\ f.$$

Where S = the total shear on the cross-section;

s = the average unit shear at the neutral axis;

b = the width of the beam at the neutral axis; and

f = the distance between the resultants of the compressive and tensile forces.

The demonstration of this proposition is as follows:

Let p = the longitudinal unit stress due to the bending moment;

R = the resultant of this stress on each side of the neutral axis;

M = the bending moment; and

a = the distance from the neutral axis to the extreme fiber.

Mr. Jonson. It is known that the shear at the neutral axis is equal to the first differential coefficient of the total longitudinal stress due to the bending moment on one side of the neutral axis.

$$s\,b = \int_{x=o}^{x=a} \int_{z=o}^{z=b} \frac{d\,p}{d\,y}\, d\,x\, d\,z \ldots\ldots (I)$$

Hence,

$$s\,b = \frac{d\,R}{d\,y} \ldots\ldots (II)$$

It is also known that the total shear on the cross-section is equal to the first differential coefficient of the bending moment.

$$S = \frac{d\,M}{d\,y} \ldots\ldots (III)$$

And, since $M = R\,f$.

$$\frac{d\,M}{d\,y} = \frac{f\,d\,R}{d\,y} \ldots\ldots (IV)$$

Hence,

$$S = \frac{f\,d\,R}{d\,y} \ldots\ldots (V)$$

By substitution, according to Equation II,

$$S = s\,b\,f \ldots\ldots (VI)$$

The formula for the maximum unit shear in a reinforced concrete beam will then be:

$$s = \frac{S}{b\,f} \ldots\ldots (VII)$$

And the formula for the stress, t, on the 45° diagonal reinforcement in one unit of length will be:

$$t = \frac{S}{f\sqrt{2}} \ldots\ldots (VIII)$$

It seems to the writer that a sharp bend at the junction of the diagonal and longitudinal reinforcements should be avoided, as it tends to produce an excessive compressive stress in the concrete at that point; and that a round bend would be better. If 1 000 lb. is allowed on the concrete and 16 000 lb. on the steel, the radius of this bend would be, approximately,

$$r = \frac{14\,A}{n} \ldots\ldots (IX)$$

Where A = the area of the diagonal, and n = the width of the same. For round rods, this would make about

$$r = 10\,d \ldots\ldots (X)$$

Where d = the diameter of the rod.

In this case, however, the stress in the diagonal would be about 40% greater at the lower end of the bend than at its upper end, so

that, instead of Equation VIII, the following formula would express the stress in the 45° diagonal reinforcement in one unit of length: Mr. Jonson.

$$t = \frac{S}{f} \quad \text{.....(XI)}$$

LEONARD C. WASON, M. AM. SOC. C. E. (by letter).—The writer was considering the submission of a paper intended to draw out a discussion leading to the adoption of a simple formula for general use in designing reinforced concrete beams, when Captain Sewell's paper appeared, therefore this discussion is submitted partly as a discussion of his paper, instead of as an independent paper. Its object is to show that sufficient data are now available to determine an accurate formula (accurate within allowable limits of variation) for general use in designing beams of rectangular or T-section, and to emphasize the fact that the best results are obtained by solving for the working instead of the ultimate strength. Therefore this is not exclusively a discussion of Captain Sewell's paper. It is hoped that all those whose formulas are compared herein, and many others, will contribute to the discussion. There are probably at least ten formulas in use besides those mentioned. The writer has discussed the subject chiefly from its commercial aspect, and has purposely omitted the more technical points relating to the disagreement in results, leaving these to be discussed by the professors, who are much better able to do so. Mr. Wason.

The need of a generally accepted method has been forced upon the writer through competition. In several cases where architects have specified spans and floor loads, and have left the design entirely open to the bidders, work has been lost because competitors have put in lighter designs. In other cases, where a bid has been submitted, accompanied by plans, and, later, the designs have been submitted for review by a consulting engineer, the cost has been increased by more than 10%, due solely to a difference in the formulas. By reference to Table 2, a summary table of comparison, it will be seen that if the work had been designed by the method proposed by the writer, and submitted to an engineer to be checked by the author's formula, the cost would have been appreciably increased, because of the much lower moment of resistance for the same section of beam. Yet structures costing many millions of dollars have been designed by the formula proposed, competition has proved it economical, and the experience of fifteen years has proved it to be safe. There is a general and simple method for designing wooden and steel beams; why should there not be one for reinforced concrete?

The general form of formulas, Equation 0, of Captain Sewell's paper, $M = u\ d\ A\ f$, is correct. The only factor about which there can be much discussion is the value of u.

Mr. Wason. The formulas which follow—proposed by nine writers—are based on different assumptions, all of which, from certain experiments, appear to have solid foundations. All are based on the same fundamental conditions, namely:

1.—All tension is carried by the steel;
2.—All compression is carried by the concrete;
3.—There is a perfect bond or union between the steel and concrete within the limits of the stresses used;
4.—The effects of shear are omitted, and failure is due to flexure only;
5.—There are no initial strains, and the same examples solved by each, using the same constants, ought to give results directly comparable.

The formulas of various writers are reduced to the same general form, and two examples (one a beam and the other a slab) are solved, first, using the same constants for all; secondly, using the constants proposed by each individual writer. The results are summarized in Table 2.

Assume a rectangular beam: span, 14 ft.; width, 12 in.; depth to center of reinforcement, 12 in.; total depth, 13½ in. Find the maximum moment of resistance, load uniformly distributed, area of steel, and neutral axis.

Assume, also, a flat slab: span, 8 ft.; width, 12 in.; depth, 4½ in.; depth to center of reinforcement, 4 in. Find the ultimate moment of resistance, load, area of steel, and position of neutral axis.

Values of Constants, and Definition of Symbols.

Broken-stone concrete. Mixture 1:3:6. Age 30 days.

$E_c = E_t$	=	modulus at working stress	3 000 000
E_s	=	modulus of steel	30 000 000
c	=	ultimate compression in concrete	2 000
t	=	ultimate tension in concrete	200
f	=	elastic limit of steel	50 000
f_1	=	working stress in steel	16 000
c_L	=	working stress in concrete compression	500

M = moment of resistance of section of beam, in inch-pounds;
W = total uniformly distributed load on beam, in pounds;
E_s = modulus of elasticity of steel;
E_c = " " " " concrete in compression;
E_t = " " " " " " tension;
f = fiber stress in steel, in pounds tension per unit of area;
c = compression in outer fiber of concrete, in pounds per unit of area;
t = tension in outer fiber of concrete, in pounds per unit of area;

Mr. Wason.

A = area of steel, in square inches;
p = ratio of area of steel to cross-section of beam.
$n = \frac{E_s}{E_c}$;
b, d, h, x, y, v, e, see Fig. 13;
z and u are ratios of d or h;
l = span of beam, in inches;
v = distance from top of beam to center of gravity of compressive stresses.

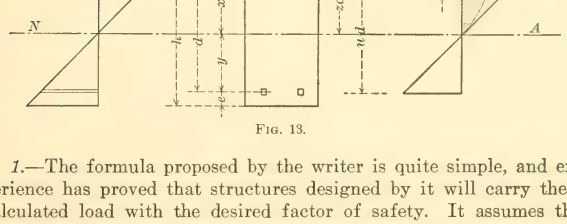


FIG. 13.

1.—The formula proposed by the writer is quite simple, and experience has proved that structures designed by it will carry their calculated load with the desired factor of safety. It assumes the neutral axis half way from the top of the beam to the center of the steel, and the center of gravity of the compressive stress at one-third of the depth from the top to the neutral axis; the compression area is confined to the upper third of the beam. The elastic theory is involved by the values selected for the working stresses of the steel and concrete, and by seeing that the areas of each are sufficient for the stresses used.*

$$A = \frac{b\,d\,c}{3\,f};$$

$$M = \frac{5}{6}\,d\,f\,A;$$

External moment, load uniformly distributed $= \frac{W\,l}{8}$.

Therefore,

$f = \frac{W\,l}{6\frac{2}{3}\,d}$, for a beam supported at its ends and uniformly loaded.

In any given case, the load and span are known. Select a convenient figure for the depth of the beam or the stress in the steel, and solve for the other unknown quantity.

*For a full demonstration, see *Transactions*, Am. Soc. C. E., Vol. XLVI, 1901, p. 102; or *Engineering Record*, September 21st, 1901, p. 272.

Mr. Wason.

Example of Beam:

$$A = \frac{b\,d\,c_1}{3\,f_1} = \frac{12 \times 12 \times 500}{3 \times 16\,000} = 1.5 \text{ sq. in.};$$

$$M = \frac{5}{6}\,d\,f\,A = \frac{5}{6} \times 12 \times 50\,000 \times 1.5 = 750\,000;$$

$$W = \frac{750\,000 \times 8}{14 \times 12} = 35\,714.$$

Example of Slab:

$$A = \frac{12 \times 4 \times 500}{3 \times 16\,000} = 0.5;$$

$$M = \frac{5}{6} \times 4 \times 50\,000 \times 0.5 = 83\,333;$$

$$W = 6\,944.$$

There is no change in constants necessary.

2.—W. K. Hatt, Assoc. M. Am. Soc. C. E., has evolved an elaborate formula for flexure.* In the form here given, it applies to rectangular beams which do not fail by shearing. The stresses in the concrete are assumed to follow a parabolic curve. The formula gives the load at the first visible crack in the concrete on the tension side. This load, according to tests by Professor Hatt, is about 20% less than that of ultimate failure.

Let $h\,z$ = the distance from the compression face to the neutral axis;

$h\,u$ = the distance from the compression face to the center of gravity of the reinforcement;

p = the ratio of the area of the steel to that of the total cross-section of the beam;

f = stress at the elastic limit of the steel.

p and u are in the control of the designer; n is fixed for the given materials and working stresses.

After a crack has formed, the neutral axis is located by the formula:

$$\frac{2}{3}\,z^2 = p\,\frac{E_s}{E_c}\,(u - z);$$

$$c = \frac{3\,p\,f}{2\,z};$$

$$M = b\,h^2\left[\frac{5}{12}\,c\,z^2 + p\,f\,(u - z)\right];$$

or, in a simpler form, where the expression in the bracket is represented by the constant, K,

$$M = b\,h^2\,K.$$

* *Engineering News*, February 27th and July 27th, 1902; and *Journal* of the Western Society of Engineers, June, 1904.

Mr. Wason.

For given conditions, a table is made for K, in order to simplify the application of the formula; or diagrams of the equation may be made and used. Professor Hatt states that, in his judgment, one-third of the amount at the first crack is the safe working moment of resistance. This would give a factor of safety, on the ultimate strength, of about $3\frac{3}{4}$.

Example of Beam :

$$p = \frac{A}{b\,h} = \frac{1.5}{12 \times 13.5} = 0.0092 ;$$

A (assumed) $= 1.5$ sq. in. ;

$$u = \frac{12}{13.5} = 0.9 ;$$

$$n = \frac{3\,000\,000}{200 \div \frac{1}{1000}} = 15 ;$$

$$\tfrac{2}{3}\, z^2 = 0.0092\, \frac{30\,000\,000}{3\,000\,000}\, (0.9 - z) ;$$

$$z = 0.29 ;$$

$$C = \frac{3 \times 0.0092 \times 50\,000}{2 \times 0.29} = 2\,379 ;$$

$$M = 12 \times 13.5^2 \left[\tfrac{5}{12} \times 2\,379 \times 0.29^2 + 0.0092 \times 50\,000\,(0.9 - 0.29) \right]$$

$$= 795\,959 ;$$

$$W = \frac{795\,959 \times 8}{14 \times 12} = 37\,906 ;$$

$$h\,z = 3.93.$$

Example of Slab :

A (assumed) $= 0.5$; p, u and z, same as above ;

$M = 88\,440$;

$W = 7\,370$.

Professor Hatt's constants :

$E_c = 4\,130\,000$; $n = 16$; $c = 250$.

Then $z = 0.345$, and $c = 2\,000$.

Beam : $M = 775\,226$;

$W = 36\,915$.

Slab : $M = 86\,136$;

$W = 7\,178$.

3.—Edwin Thacher, M. Am. Soc. C. E., proposes the following formula.* In the form here given, it is modified to apply to any width, b, of beam instead of a beam 1 in. wide.

Let f = the stress per square inch on the steel, the gross area = the ultimate strength per square inch of the test piece + 10%;

* *Transactions*, Assoc. of C. E. of Cornell Univ., for 1902.

Mr. Wason. E_c = the modulus under a pressure of from 1 000 to 2 000 lb. per sq. in.;

$$y = \frac{d}{\left(\frac{c}{f} \times \frac{E_s}{E_c} + 1\right)};$$

$$x = d - y;$$

$$A = \frac{d\,b}{2\left[\frac{f}{c} + \left(\frac{f}{c}\right)^2 \times \frac{E_c}{E_s}\right]};$$

$$M = \frac{f}{3}\left[\frac{E_c}{E_s}\frac{x^3}{y}\,b + 3\,A\,y\right];$$

$$W = \frac{24\,f}{91}\left[\frac{E_c}{E_s} \times \frac{x^3}{y}\,b + 3\,A\,y\right],$$

for a load uniformly distributed.

To design a beam, assume values of f, E_s, E_c, c, b and d. For other systems of loading or of support, the coefficients in the equations for M and W, outside the bracket, would change.

Example of Beam:

$$y = \frac{12}{\frac{2\,000}{50\,000} \times \frac{30\,000\,000}{3\,000\,000} + 1} = 8.57;$$

$$x = 3.43.$$

$$A = \frac{12 \times 12}{2\left[\frac{50\,000}{2\,000} \times \frac{(50\,000)^2}{2\,000} \times \frac{3\,000\,000}{30\,000\,000}\right]} = 0.82;$$

$$M = \frac{50\,000}{3} \times \left[\frac{3\,000\,000}{30\,000\,000} \times \frac{3.43^3}{8.57} \times 12 + 3 \times 0.82 \times 8.57\right]$$

$$= 445\,499;$$

$W = 21\,214.$

Example of Slab:

$y = 2.86$; $x = 1.14$; $A = 0.274$; $M = 49\,500$; $W = 4\,125$.

Mr. Thacher's units:

Factor of safety, $3\frac{1}{2}$.

$E_c = 1\,220\,000$; $c = 2\,050$; $f = 64\,000 + 10\%$;

Beam: $A = \frac{b\,d}{165} = \frac{12 \times 12}{165} = 0.873$;

$M = 30.62\,d^2 \times 12 \times 12 = 634\,936$;

$W = 30\,235$;

$$y = \frac{12}{\frac{2\,050}{70\,400} \times \frac{30\,000\,000}{1\,220\,000} + 1} = 7.01;$$

$x = 4.99$;

Slab: $A = 0.291$; $y = 2.33$; $x = 1.67$; $M = 70\,548$; $W = 5\,879$.

4.—William H. Burr, M. Am. Soc. C. E., gives these formulas* for rectangular beams for the special case where the tension in the concrete is neglected and the steel is on the tension side only:

* Mr. Wason.

$$x = -\frac{E_s A}{E_c b} \pm \sqrt{\left(\frac{E_s A}{E_c b}\right)^2 + 2\,\frac{E_s A}{E_c b}\,d} = \frac{E_s}{E_c} \times \frac{d - x}{x}\,c = 34\,920;$$

$$M = c\left[\frac{b\,x^2}{3} + \frac{E_s}{E_c} \times \frac{A}{x}\,(d - x)^2\right].$$

Example of Beam:

A (assumed) $= 1.5$;

$$x = -\frac{30\,000\,000 \times 1.5}{3\,000\,000 \times 12} \pm \sqrt{1.25^2 + 2 \times 1.25 \times 12} = 4.37;$$

$$M = 2\,000\left[\frac{12 \times 4.37^2}{3} + \frac{30\,000\,000 \times 1.5}{3\,000\,000 \times 4.37}(12 - 4.37)^2\right] = 552\,180;$$

$W = 26\,294.$

Example of Slab:

A (assumed) $= 0.5$;

$$x = -0.417 \pm \sqrt{0.417^2 + 2 \times 0.417 \times 4} = 1.45;$$

$M = 61\,640$;

$W = 5\,137.$

Professor Burr's units:

$$c = 3\,100.$$

Beam: $x = 4.38$; $M = 855\,879$; $W = 40\,756.$

Slab: $x = 1.45$; $M = 95\,480$; $W = 7\,957.$

5.—A. L. Johnson, M. Am. Soc. C. E., proposes the following:

f = elastic limit of the steel;

$$y = \frac{2\,f\,E_c}{3\,c\,E_s}\,x; \quad x + y = d;$$

$$A = \frac{75\,c\,b\,x}{120\,f};$$

$$M = f\,A\left(y + \frac{2\,x}{3}\right).$$

Example of Beam:

$$y = \frac{2 \times 50\,000 \times 3\,000\,000}{3 \times 2\,000 \times 30\,000\,000}\,x = 1.67\,x;$$

$$x + y = 12;$$

$$x = 4.50;$$

$$y = 7.50;$$

$$A = \frac{75 \times 2\,000 \times 12 \times 4.5}{120 \times 50\,000} = 1.35;$$

* "The Elasticity and Resistance of the Materials of Engineering," 1903 edition.

Mr. Wason.

$$M = 50\,000 \times 1.35 \left(7.50 + \frac{2 \times 4.5}{3}\right) = 708\,750;$$

$$W = 33\,750.$$

Example of Slab:

$$y = 2.5; \quad x = 1.5; \quad A = 0.45;$$

$$M = 78\,750; \quad W = 6\,563.$$

Mr. Johnson's units:

$$E_s = 29\,000\,000; \quad y = 1.72\,x; \quad x = 4.41; \quad A = 0.0195\,b\,x = 1.32;$$

$$M = 2\,750\,b\,x^2 = 641\,850; \quad W = 30\,564.$$

Slab: $x = 1.47$; $A = 0.34$; $M = 71\,280$; $W = 5\,940$.

6.—J. Kahn, Assoc. M. Am. Soc., C. E., uses a compressive area different from that used by any other writer. See Fig. 14.

$f =$ the ultimate tensile stress in the steel;

$$y = \frac{15\,A + b\,d^2}{30\,A + 2\,b\,d};$$

$$M = f\,A\left(\frac{5}{8}\,x + y\right);$$

b should never be less than $\frac{A\,f}{1\,800\,x}$.

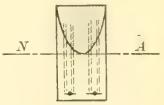


FIG. 14.

Example of Beam:

$$y = \frac{15 \times 1.5 + 12 \times 12^2}{30 \times 1.5 + 2 \times 12 \times 12} = 5.26;$$

A (assumed) $= 1.5$;

$$M = 50\,000 \times 1.5 \left(\frac{5}{8} \times 6.74 + 5.26\right) = 710\,250;$$

$$W = 33\,821.$$

Example of Slab:

A (assumed) $= 0.5$; $y = 1.80$; $M = 79\,375$; $W = 6\,614$.

Mr. Kahn's units:

$$E_c = 2\,000\,000; \quad c = 3\,000; \quad f = 64\,000;$$

A (assumed) $= 1.5$; $y = 5.26$;

$$M = 64\,000 \times 1.5 \left(\frac{5}{8} \times 6.74 + 5.26\right) = 909\,120;$$

$$W = 43\,291.$$

Slab: $y = 1.80$; $M = 101\,600$; $W = 8\,467$.

7.—A. N. Talbot, M. Am. Soc. C. E., proposes the following,* which is the special case for the ultimate deformation of concrete:

Let $p = \frac{A}{b\,d}$ = ratio of area of metal reinforcement to area of concrete above center of reinforcement;

* *Engineering Record*, August 13th, 1904; and *Journal* of Western Society of Engineers, August, 1904.

Mr. Wason.

$$z = \sqrt{\frac{2\,p\,n}{1 - \frac{1}{3}} + \frac{p^2\,n^2}{(1 - \frac{1}{3})^2}} - \frac{p\,n}{1 - \frac{1}{3}};$$

$$v = \frac{3}{8}\,z\,d;$$

$$M = A\,f(d - v).$$

The solution of the values of z and v for smaller stresses than the ultimate is somewhat complicated. However, using the values of z obtained by tests of 1:3:6 concrete beams, the formula becomes:

$$M = A\,f\,(0.906 - 6.5\,p)\,d.$$

This is a simple and convenient form to use.

Example of Beam:

A (assumed) $= 1.5$;

$$p = \frac{1.5}{12 \times 12} = 0.01;$$

$$n = \frac{30\,000\,000}{3\,000\,000} = 10;$$

$$z = \sqrt{\frac{2 \times 0.01 \times 10}{\frac{2}{3}} + \frac{0.01^2 \times 10^2}{\left(\frac{2}{3}\right)^2}} - \frac{0.01 \times 10}{\frac{2}{3}} = 0.418;$$

$$v = \frac{3}{8} \times 0.418 \times 12 = 1.88;$$

$M = 1.5 \times 50\,000 \times (12 - 1.88) = 759\,000$;

$W = 36\,143$.

Example of Slab:

$v = 0.63$;

$M = 84\,325$;

$W = 7\,027$.

Professor Talbot's units:

E_c = initial modulus average, 2 000 000; $n = 15$; $z = 0.482$; $v = 2.17$; $M = 1.5 \times 50\,000 \times 9.83 = 737\,250$; $W = 35\,107$.

Slab: $v = 0.72$; $M = 82\,000$; $W = 6\,833$.

8.—John S. Sewell, M. Am. Soc. C. E.

$$x + y = d;$$

$$y = \frac{f_1\,E_c}{c_1\,E_s}\,x;$$

$$y = \frac{f\,E_c}{0.8\,c\,E_s}\,x;$$

$$v = 0.64 + y.$$

Let $c_1 = 0.8\,c$;

$x - v = 0.64\,x$;

area under curve $= 0.57\,c_1\,x = 0.456\,c\,x$;

$0.465\,b\,c\,x = A\,f$;

$$M = 0.456 \times 0.64\,b\,c\,x^2 + A\,f\,y = 0.292\,b\,c\,x^2 + A\,f\,y.$$

Mr. Wason.

Example of Beam:

$$y = \frac{50\,000 \times 3\,000\,000}{0.8 \times 2\,000 \times 30\,000\,000}\,x = 3.125\,x;$$

$$x = 2.91; \quad y = 9.09; \quad u = 10.95;$$

$$A = \frac{0.456 \times 12 \times 2\,000 \times 2.91}{50\,000} = 0.637;$$

$$M = 0.292 \times 12 \times 2\,000 \times 2.91^2 + 0.637 \times 50\,0000 \times 9.09 = 348\,860;$$

$$W = 16\,612.$$

Example of Slab:

$$x = 0.97; \quad y = 3.03; \quad A = 0.212; \quad u = 3.65;$$

$$M = 0.292 \times 12 \times 2\,000 \times 0.97^2 + 0.212 \times 50\,000 \times 3.03 = 38\,706;$$

$$W = 3\,225.$$

Captain Sewell's units:

$$f = 45\,000; \quad c = 2\,500; \quad \frac{E_c}{E_s} = \frac{1}{15}.$$

Beam:

$$y = \frac{45\,000 \times 1}{0.8 \times 2\,500 \times 15}\,x = 1.5\,x; \quad x = 4.8; \quad y = 7.2; \quad u = 10.27;$$

$$A = \frac{0.456 \times 12 \times 2\,500 \times 4.8}{45\,000} = 1.46;$$

$$M = 0.292 \times 12 \times 2\,500 \times 4.8^2 + 1.46 \times 45\,000 \times 7.2 = 674\,870;$$

$$W = 32\,137.$$

Slab:

$$x = 1.6; \quad y = 2.4; \quad A = 0.486; \quad u = 3.42;$$

$$M = 0.292 \times 12 \times 2\,500 \times 1.6^2 + 0.486 \times 45\,000 \times 2.4 = 74\,914;$$

$$W = 6\,243.$$

9.—Mr. F. D. Warren has written a handbook * on reinforced concrete, in which there are a great many tables. In order to determine their value, the formulas based on the method of the late J. B. Johnson. M. Am. Soc. C. E., published in *Engineering News* in 1895, is submitted for review. An area of concrete equal to ten times the area of the steel is put in the same plane. The moment of inertia of this section is found, and the location of the neutral axis.

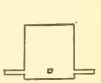


Fig. 15.

$$\frac{E_s}{E_c} = 10;$$

M_o = working moment, factor of safety, $3\frac{1}{2}$;

$$\frac{M_o}{500} = \frac{1\,b\,d^3}{4\,d}$$

Preliminary step to find stress in compression in concrete; with factor of safety of 3.5 and $c = 3\,000$.

* "A Handbook on Reinforced Concrete," pp. 78-81.

$$\frac{M_o}{f_1} = \frac{A\ h^2}{h}$$

Preliminary step to find area of steel; neutral axis assumed to be from 1.5 to 2.0 in. below the center of gravity of the section. Mr. Wason.

Transpose this area of steel into an area of concrete, and solve for the moment of inertia and the position of the neutral axis, neglecting the area of the concrete below the plane of the steel. If the neutral axis differs materially from 1.5 to 2.0 below the center of gravity, use this value to determine h in the formula, $\frac{M_o}{f_1} = A\ h$, and with the new value again solve for A. Then check the fiber stress of the concrete in compression.

Example of Beam:

$\frac{M_o}{333} = \frac{1\ b\ d^3}{4\ d}$, when $c = 2\,000$;

$M_o = 83.33\ b\ d^2 = 83.33 \times 12 \times 12^2 = 144\,000$, working load;

$M = 144\,000 \times 3.5 = 504\,000$, ultimate load;

$\frac{M}{f} = \frac{A\ y^2}{y}$;

y (assumed) $= 4.5$;

$A = \frac{M}{f\ y} = \frac{504\,000}{50\,000 \times 4.5} = 2.24.$

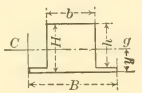


FIG. 16.

Assume 1-in. bars.

$y\ [B\,H - h\,(B - b)] = \frac{1}{2}\ B\ H^2 - h\ (B - b)\ (H - \frac{h}{2})$;

$y\ [34.4 \times 12.5 - 11.5\ (34.4 - 12)]$

$= 0.5 \times 34.4 \times 12.5^2 - 11.5\ (34.4 - 12)\ (12.5 - \frac{11.5}{2})$;

$y = \frac{748.7}{172.4} = 4.34$;

$x = 8.16$, or 2.16 below the central axis;

$I = \frac{12 \times 12^3}{12} + 12 \times 12 \times 2.16^2 = 2\,400$;

$\frac{I}{x} = \frac{2\,400}{8.16} = 294$;

$c = \frac{504\,000}{294} = 1\,714$;

$A = \frac{M}{f\ y} = \frac{504\,000}{50\,000 \times 4.34} = 2.32.$

Example of Slab:

$M = 83.33 \times 3.5 \times 12 \times 4^2 = 56\,000$;

Mr. Wason.

$$A = \frac{56\,000}{50\,000 \times 1.5} = 0.75;$$

y (assumed) $= 1.5$.

Assume $\frac{1}{2}$-in. bars.

$$y\,[19.5 \times 4.25 - 3.75\,(19.5 - 12)] = 0.5 \times 19.5 \times 4.25^2 - 3.75\,(19.5 - 12)\left(4.25 - \frac{3.75}{2}\right);$$

$$y = \frac{109.7}{54.88} = 2.00;$$

$x = 2.25$, or 0.25 below the central axis:

$$I = \frac{12 \times 4^3}{12} + 12 \times 4 \times 0.25^2 = 68.0;$$

$$\frac{I}{x} = 30.7;$$

$$A = \frac{56\,000}{50\,000 \times 2.0} = 0.56.$$

F. D. Warren's constants:

Beam: $c = 3\,000$; $f = 53\,000$; $f_1 = 15\,000$.

Factor of safety $= 3.5$.

$M_o = 125\,b\,d^2$;

$M = 125\,b\,d^2 \times 3.5 = 125 \times 12^2 \times 12 \times 3.5 = 756\,000$;

$$A = \frac{M}{f\,y} = \frac{756\,000}{53\,000 \times 4} = 3.57;$$ y (assumed) $= 4$; 1-in. bars assumed

$$y\,[47.7 \times 12.5 - 11.5\,(47.7 - 12)] = 0.5 \times 47.7 \times 12.5^2 - 11.5\,(47.7 - 12)\left(12.5 - \frac{11.5}{2}\right);$$

$$y = \frac{955.3}{185.7} = 5.14;$$

$x = 7.36$, or 1.36 below the central axis;

$$I = \frac{12 \times 12^3}{12} + 12 \times 12 \times 1.36^2 = 1\,994.6;$$

$$\frac{I}{x} = 271;$$

$$c = \frac{756\,000}{271} = 2\,790;$$

$$A = \frac{756\,000}{53\,000 \times 5.14} = 2.78.$$

Slab:

$M = 125 \times 12 \times 4^2 \times 3.5 = 84\,000$;

$$A = \frac{84\,000}{53\,000 \times 1.75} = 0.906;$$

y (assumed) $= 1.75$;

Mr. Wason.

$y\,[21 \times 4.25 - 3.75\,(21 - 12)]$

$$= 0.5 \times 21 \times 4.25^2 - 3.75\,(21 - 12)\left(4.25 - \frac{3.75}{2}\right);$$

$$y = \frac{109.6}{55.5} = 1.98;$$

$x = 2.27$, or 0.27 below the central axis;

$$A = \frac{84\,000}{53\,000 \times 19.8} = 0.80.$$

An examination of the results in Table 2 will show that in three of the nine there is a fairly close agreement with each other, whether with the use of the same constants for all, or the constants recommended by each author, although the assumptions of each are quite different. The three which agree fairly well are: Wason, Hatt, and Talbot. These three, then, would appear to be the most general in character, and the writer's method gives the safest result with the same constants. Professor Talbot's analysis appears to be the most rational solution of the problem, from a profoundly scientific standpoint.

Without doubt, the stress diagram of the concrete in compression is a curve somewhat resembling a parabola. No allowance for tension of concrete should be made. There seems to be, among many writers, a lack of appreciation of the fact that results obtained by solving for the ultimate strength of a reinforced concrete beam, then dividing by a given factor of safety, or using the corresponding working stresses to solve for the working strength of a beam, do not give the same results, or results which can be compared. To illustrate: Take the beam previously considered, and the ultimate strength of the concrete as the force, instead of the area of the steel, with the parabolic treatment of the compression, and the neutral axis at one-half the depth to the center of the reinforcement.

Then, arm of ultimate moment $= \frac{5}{8} \times \frac{1}{2}\,d + \frac{1}{2}\,d = 0.8125\,d$;

compressive force of concrete $= \frac{5}{8}\,c \times \frac{1}{2}\,d\,b = 0.3125\,c\,d\,b$;

$M = 0.254\,c\,d^2\,b = 0.254 \times 2\,000 \times 12 \times 12^2 = 877\,824$;

$\frac{1}{4}\,M =$ working moment;

$M_o = 219\,456$.

If we use an outside fiber stress of one-quarter of the ultimate, namely, $c^1 = 500$, and solve for the working moment of resistance with the triangular, straight-line treatment of compression (see Fig. 17), then,

Arm of working moment $= \frac{2}{3} \times \frac{1}{2}\,d + \frac{1}{2}\,d = 0.833\,d$;

Mr. Wason.

compressive force of concrete $= \frac{1}{2}\ c^1 \times \frac{1}{2}\ b\ d = 0.25\ c^1\ b\ d$;

$M_0 = 0.208\ c^1\ b\ d^2 = 180\,000$.

The correct method is to solve for the working strength. Assuming a working compressive stress of one-fourth the ultimate, the difference between the straight line and the parabola is negligible, and the triangular area can be used, as it is simpler. The variation in the quality of the best concrete will more than offset the refinement of retaining the curve. If the triangular area is used for ultimate compression, the result will be too large, in the ratio of $\frac{1}{2}$ to $\frac{5}{16}$, or $37\frac{1}{2}$ per cent. The modulus of elasticity is used only in finding the position of the neutral axis, in order to find the center of gravity of the compressive force. The position of the neutral axis is of very little value.

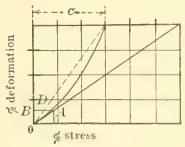


FIG. 17.

FIG. 18.

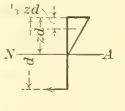


FIG. 19.

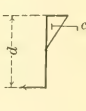


FIG. 20.

To illustrate, take two extreme cases: First, $E_c = 4\,500\,000$ and $\frac{1}{2}\%$ of steel; secondly, $E_c = 1\,250\,000$ and 2% of steel. Then, by Talbot's formula: First case, $z = 0.266$; second case, $z = 0.616$.

Arm of moment $= d - \frac{1}{3}\ z\ d$

First case " " " $= 0.925\ d$

Second case " " " $= 0.795\ d$

Mean..... $= 0.86\ d$

The difference is $0.13\ d$ or 14 per cent. If a factor of safety of 4 is used on the crushing strength of concrete, then the 14% of the ultimate strength is only $3\frac{1}{2}\%$ of the working strength. Greater differences are frequently found among several test specimens made from the same batch. As the case cited covers a wide range, from a rich mixture at an age of several months, with insufficient reinforcement, to a lean mixture at the age of one month, with an excess of metal, with an error negligible in common practice, it is evident that no change is necessary in the formula for any ordinary cases of construction, because they always fall within these limits. This adds greatly to the simplicity of making designs, and to universality in the use of tables.

TABLE 2.—SUMMARY OF RESULTS USING THE SAME CONSTANTS:

FOR BEAMS.

Author.	Ultimate bending moment, in pounds.	Breaking load, in pounds.	Area of steel, in square inches.	Stress in steel, in pounds per square inch.	Distance, top to neutral axis, in inches.	Arm of moment of resistance, in inches.	Arm, as percentage of d.
Wason.....	750 000	35 714	1.5	50 000	6.00	10.00	0.833
Hatt.....	795 939	37 906	1.5	50 000	3.93	10.53	9.877
Thacher....	445 499	21 214	0.82	50 000	3.43	10.86	0.905
Burr.....	552 180	26 294	1.50	34 920	4.37	10.54	0.878
Johnson....	708 750	33 750	1.35	50 000	4.50	10.32	0.86
Kahn.....	710 250	33 821	1.50	50 000	6.74	9.47	0.789
Talbot.....	759 000	36 143	1.50	50 000	5.02	10.12	0.843
Sewell.....	348 860	16 612	0.64	50 000	2.91	10.95	0.913
Warren.....	594 000	24 000	2.24	50 000	8.16	9.28	0.773

FOR SLABS.

Wason.....	83 333	6 944	0.5	50 000	2.00	3.33	0.833
Hatt.....	88 440	7 370	0.5	50 000	1.31	3.51	0.877
Thacher....	49 500	4 125	0.28	50 000	1.14	3.62	0.905
Burr.....	61 640	5 137	0.5	34 920	1.45	3.52	0.878
Johnson....	78 750	6 563	0.45	50 000	1.50	3.44	0.86
Kahn.....	79 375	6 614	0.5	50 000	2.20	3.18	0.795
Talbot.....	84 325	7 027	0.5	50 000	1.67	3.37	0.842
Sewell.....	38 706	3 225	0.21	50 000	0.97	3.65	0.913
Warren.....	56 000	4 667	0.57	50 000	2.25	3.25	0.812

SUMMARY OF RESULTS USING EACH AUTHOR'S CONSTANTS:

FOR BEAMS.

Wason.....	750 000	35 714	1.5	50 000	6.00	10.00	0.833
Hatt.....	775 226	36 915	1.5	50 000	4.66	10.25	0.854
Thacher....	634 936	30 235	0.87	70 400	4.99	10.33	0.861
Burr.....	855 879	40 756	1.5	53 940	4.38	10.54	0.878
Johnson....	941 850	39 564	1.32	50 000	4.41	10.35	0.863
Kahn.....	909 120	43 291	1.5	64 000	6.74	9.47	0.789
Talbot.....	737 250	35 107	1.5	50 000	5.78	9.83	0.819
Sewell.....	674 870	32 137	1.46	45 000	4.8	10.27	0.856
Warren.....	756 000	36 000	2.78	53 000	7.36	9.55	0.792

FOR SLABS.

Wason.....	83 333	6 944	0.5	50 000	2.00	3.33	0.833
Hatt.....	86 136	7 178	0.5	50 000	1.55	3.42	0.855
Thacher....	70 548	5 875	0.29	70 400	1.67	3.44	0.86
Burr.....	81 933	6 828	0.5	53 940	2.15	3.28	0.82
Johnson....	71 280	5 940	0.34	50 000	1.47	3.45	0.862
Kahn.....	101 600	8 467	0.5	64 000	2.20	3.18	0.795
Talbot.....	82 000	6 833	0.5	50 000	1.93	3.28	0.82
Sewell.....	74 914	6 243	0.49	45 000	1.6	3.42	0.855
Warren.....	84 000	7 000	0.80	53 000	2.27	3.24	0.810

Mr. Wason. In a large number of tests examined, the neutral axis for one-quarter of the ultimate load has been found to be not far from one-half the depth from the top of the beam to the center of reinforcement. It is sometimes above and sometimes below. To assume it half way, agrees very closely, therefore, with the observed facts. With this position and a straight-line distribution within the working stress, the center of gravity of the compressive stresses is two-thirds of the distance up from the neutral axis to the top of the beam. Therefore, the arm of the moment of resistance is $\frac{5}{6} d = 0.833\ d$.

The method of designing beams with a T-section need not add complications. The foregoing formula can be applied without difficulty. The same value of u can be used as for a rectangular beam, inasmuch as the center of gravity of the compressive forces for the T-section, usually, will nearly coincide with that for the rectangular one, and if it does not, u will be less than its true value, therefore the beam will be somewhat heavier than is necessary. If the depth and size of the steel are fixed, it is then merely necessary to find a sufficient area of concrete to balance this; or take the maximum allowable width of flange, which usually may be taken as four times the width of the web, and from the other known dimensions of the section, the area in compression may be obtained. Knowing the ratio of the allowable working stresses in the concrete and the steel, the area of the steel is fixed. One advantage of this formula is that the area of the concrete above the neutral axis multiplied by its average stress may be substituted for the area of the steel.

In the writer's practice, he has never known a case where a floor was composed of concrete leaner than 1:3:6, or richer than 1:2:4, and as a designer will generally use one mixture at all times, there is no complication by a large number of ratios of areas of steel to concrete, as the percentage is a fixed amount determined entirely by the working stress of each material. In using such a formula, it is as easy to check existing work designed by others as to design new work independently by it.

In using the method proposed by Professor Hatt, there are too many variables depending upon one another. The position of the neutral axis varies with the percentage of the area of reinforcement to that of the concrete; the ratio of moduli of elasticity of the two materials, which in turn varies with the mixtures, age, and character of the cement, sand and stone; and the stress-strain curve differs for each case. Moreover, the position of the neutral axis is of little consequence. Its position is not accurately known at the ultimate strength of the beam, therefore the value of c, corresponding with a given f, is only approximate. The location of the first crack is also too variable to be a safe guide to the proper working load. He places the steel too near the bottom of the beam.

Mr. Wason.

The method proposed by Mr. Thacher appears to be in error in finding the value of A, which is carried into the solution of the moment of resistance. The writer has never found the percentage of steel per inch of width of beam useful in designing structures.

Professor Burr places the ultimate strength of the concrete too high, and does not get full efficiency from the steel.

Mr. A. L. Johnson stakes his reputation and his design on the fact that reinforced concrete stretches more than plain concrete; to quote his own words:*

"As stated in the introduction, this effect of the embedded metal upon the extensibility of the concrete is the quality upon which the whole art of steel-concrete construction rests. Fortunately, it is a quality as to the existence of which there can be no doubt. The art had been a success for many years before science stepped in to tell us why."

Mr. Kahn's method the writer has been unable to check by any of the ordinary principles of design, and he sees no logical reason for the area used in compression.

Professor Talbot's formula is the most rational of any yet proposed, covering every point, and it can be readily applied to any degree of loading. It is in the form proposed for the universal formula.

Captain Sewell bases his determination for the position of the neutral axis upon the ratio of the moduli of elasticity and the working stress of the two materials. As these do not vary proportionally, results based upon this method are inaccurate. Moreover, with concrete wherein the modulus and stress are not proportional, the assumption that a plane before bending is a plane after bending is open to doubt. He states that the values of c and E_c must always be those that correspond to each other. The value of c differs somewhat with different brands of cement, and, as the designer does not know what brand will be used in his work, the correct ratio of c to E_c may not be used, and a safe value for all brands may not be economical. The writer's study of recent tests at the Watertown Arsenal led chiefly to the conclusion that it is best not to use a certain brand which was largely used in making these tests. The ultimate strength of several brands varied widely, yet the modulus at the same stress, approaching the ultimate load, varied but little. This is somewhat similar to the variation in the elastic limit and ultimate strength of steel with a change in the percentage of carbon, while the modulus of elasticity remains constant. This is a serious defect in the formulas involving ratios of these stresses.

Mr. Warren's method is a series of approximations, and the result is the same. Moreover, using his method involves a longer task

* Catalogue of St. Louis Expanded Metal Fireproofing Company, 1903, p. 53.

Mr. Wason. than any of the others. The result shows an extravagant use of steel and a lack of efficiency in the concrete. The stress in this material is not as great as his formula would indicate. A thickness of 1 in. of concrete below the steel is not enough when the size of the bar exceeds $\frac{3}{4}$ in. square.

Some engineers solve for the working stress of the beam, and use the parabolic curve for the compressive stresses. This is inaccurate, inasmuch as, within the working stresses, the stress is almost exactly proportional to the strain, so that a plot is either a straight line or its difference from a straight line is negligible. With this method of design, the outside fiber stress in compression is actually about $1\frac{1}{2}$ times as great as it is assumed, as shown below.

For a parabolic curve: $c = 500$; $p = 0.01$; $n = 10$; $x = 0.42$.

$$\frac{M}{b\,d^2} = \frac{2}{3}\,c\,x\left(1 - \frac{3}{8}\,x\right) = \frac{2}{3} \times 500 \times 0.42\left(1 - \frac{3}{8} \times 0.42\right)$$
$$= 140\,x \times 0.843 = 118.$$

For a straight line: $c = 750$; $p = 0.01$; $n = 10$; $x = 0.36$.

$$\frac{M}{b\,d^2} = \frac{c}{2}\,x \times \left(1 - \frac{x}{3}\right) = \frac{1}{2} \times 750 \times 0.36\left(1 - \frac{0.36}{3}\right)$$
$$= 135 \times 0.88 = 118.8.$$

x is taken from diagrams, using p as ordinates and x as abscissas for given values of n, x being deduced from the formula for a parabola:

$$x = -\frac{3}{2}\,n\,p + \sqrt{\left(\frac{3}{2}\,n\,p\right)^2 + 3\,n\,p};$$

and for a straight line:

$$x = -\,n\,p + \sqrt{(n\,p)^2 + 2\,n\,p}.$$

These plots were made by Professor Arthur W. French. It will be seen that the results are almost identical, although $c = 500$ in one case and 750 in the other.

Captain Sewell's taking 80% of the ultimate strength as a maximum stress on the concrete when the steel reaches its elastic limit is apparently based on Professor Hatt's tests at the first crack. The working stress obtained in concrete on this basis, using $2\frac{1}{2}$ as a factor of safety, or 32% of the ultimate strength, seems to be reasonable. The writer would recommend using the sum of the dead and live loads as the load on which the factor is based, except in the rare cases where the dead load is larger than the live load, when (live load + $\frac{4}{10}$ dead load) $\times\ 2\frac{1}{2}$ = the elastic limit of the steel, may be used. The limit of elastic deformation determined by Professor Bauschinger, which the writer has never seen questioned, was about seven-tenths of the ultimate resistance; therefore, approxi-

mately, the above factor of safety is based on the elastic limit of the concrete as well as of the steel. Mr. Wason.

In designing, it is assumed that there are no initial strains in the composite structure, but, nevertheless, they exist, as the shrinkage of the cement in setting produces an initial compression in the steel which the formula does not recognize, and this, if neglected, adds to the factor of safety. Inasmuch as the amount of stress cannot be accurately determined, this is the wiser course.

Captain Sewell states that beams designed by the right line are heavier than necessary, and that the parabola rather inclines to the opposite extreme. When solving for the working stresses, this is of less consequence. His value, $u = 0.85$, agrees quite closely with the mean of the two extreme cases previously solved by Professor Talbot's formula, and is sufficiently accurate to be generally used if the curved stress-strain principle be accepted as correct. The writer believes it is not correct when working stresses are considered.

The elaborate discussion as to the most economical design, considering the cost of the steel and the concrete, is very interesting and suggestive, but, inasmuch as the cost of the two materials is entirely independent of their ratio of stresses in practical application, the writer has found that it is sufficiently accurate and economical to design beams with the maximum allowable depth, in order to obtain the maximum of economy.

From every test which has come to the writer's attention, he is convinced that there is no economy in reinforcing the beam with steel against compression, and is glad to see this opinion confirmed. There is not as great economy in using it in compression as in tension, but the writer would go still further and say, from his present knowledge, that it is never advisable to use steel in compression in beams. The tests made for him at the Massachusetts Institute of Technology showed no value whatever in top reinforcement, and to rely on it would be dangerous.

There are other reasons to prevent a theoretically accurate design from being actually carried out on the work. As the ultimate strength and modulus of elasticity increase with age, while the steel does not, the design of the beam becomes unbalanced, thus changing the ratio of the moduli of the two materials, the position of the neutral axis, and, especially important, the ultimate compressive stress, thus changing the expected result from any design where these items are used as factors. Actual transverse tests, where the neutral axis has been located, show that the outside fibers sustain a greater stress than in columns uniformly compressed.

On work of any size, the concrete is mixed by machinery, while laboratory tests are almost invariably mixed by hand. Samples of hand-mixed concrete from actual work give better results than labo-

Mr. Wason. ratory specimens. In two sets of tests, reported at the Watertown Arsenal in 1897 and 1900, the difference in strength between hand-mixed and machine-mixed concrete, where both cases were exceptionally well done, was 11% in favor of the machine-mixed. In the other set, under commercial conditions, the difference was 25% in favor of the machine-mixed. There is quite a difference in the modulus of machine-mixed and hand-mixed concrete. Wet or plastic mixtures thoroughly or lightly tamped affect the density and the bond with steel.

The bond or union between the steel and the concrete increases with age. The adhesion does not reach a very high value in one month, but increases steadily beyond this point for a number of months. Thus, a design based on adhesion which would not be safe at one month would be safe at a greater period. A mechanical bond is always to be preferred to simple adhesion, however. While the formulas given take no notice of this, the designer must, before completing his work.

Fortunately, all these variations are in the direction of safety, but when natural causes, of which the most elaborate formula can take no cognizance, produce such differences, why attempt great refinement, especially when the values of the several factors cannot be determined with absolute precision?

The discussion resolves itself into:

1. Should the ultimate or the working stresses be used;

2. What arm should be given to the internal moment of resistance, the formula to be used being in the form:

$$M = u\ d\ A\ f.$$

The arm proposed by the writer, $\frac{5}{6}\ d$, or 0.833 d, is 1.7% safer than that proposed by Captain Sewell, and he believes it is more nearly correct.

There are many rules in practice for the quantity of concrete necessary to embed the steel in the bottom of a beam. The practice followed by the writer, which has proved entirely satisfactory in twelve years of practice, is to use a quantity equal to twice the diameter of the bar below its center. In work of exceptional importance, or where there is unusual danger from exposure to fire, this may be increased to $2\frac{1}{2}$ times the diameter of the bar, and this will be found to be ample in the worst cases. The width of the beam should be at least equal to 3 times the diameter of the bar. If this is used as a minimum, there is danger in some cases of shear cracks. Adding an inch to the amount above given, or, at most, 4 times the section of the bar, will be found sufficient for beams reinforced with a single bar. Where more than one is used, allow a clear space between the bars equal to their diameter.

Mr. Wason.

A good deal has been said by various writers about the percentage of steel to use. The limiting factor is the area of the concrete. After once determining the working stress of this, the percentage of steel is fixed for any given stress, and this ratio is a constant.

Among the advantages of solving for the working stress, in addition to those previously advanced, is the fact that a great many tests of the crushing strength of concrete of different mixtures and brands of cement have been made when measurements have not been taken to determine its elastic quality. A general average of all these is easy to obtain, or an average omitting the exceptionally high ones, and this may be used as the ultimate strength. In obtaining results of this character, there is no danger of misinterpreting their meaning. The variations in the actual results from those theoretically expected are well within the limits of allowable error. The advantage in commercial practice of all work of different designers agreeing in their principle with one another is as important as in designs made for wood and steel.

In all the foregoing, nothing has been said about web stresses. When the writer began constructing this class of work, twelve years ago, he did not know enough to reinforce the beams against shear. There are a good many structures without any reinforcement which have proved entirely satisfactory in actual service. On account of this experience, he still continues to build many floors for light loads without any web reinforcement. There is one building, however, in which the floors are habitually heavily overloaded by the shock of falling weights. In this, some beams have been cracked, which would indicate that there is not a very large factor of safety, and therefore reinforcing against web stresses is desirable. From the experience of actual structures without web reinforcement, and from laboratory tests, it is evident that the concrete will withstand a considerable part of the web stresses, enabling the beam to develop nearly the ultimate strength of its tension and compression members. The writer dissents, therefore, from Captain Sewell's opinion that concrete should no more be relied upon for connections than for resisting tensile flange stresses. Especially is this true when economy is considered.

When beams are to carry a quiet load, or one with but little vibration, the concrete can be allowed to resist shear to the amount of 80 lb. per sq. in. But when there is heavy vibration or sudden shocks, reinforce for the entire web stress. On account of the frequent lapse of time between filling a beam and spreading the panel or flange portion of the floor, it is advisable to use stirrups bent to extend at least 1 ft. each way from the beam into the panel to bond the two together. This is seldom done. It is most needed at the center of the span. Deformed bars are far superior to plain ones, and, now

Mr. Wason. that there is little difference in price between them, there is no excuse whatever for the use of plain bars.

In comparative tests, where some of the main bars were bent diagonally upward, reaching the top of the beam near the support, the greatest strength was obtained—25% greater than with the use of diagonal stirrups attached rigidly to the main bars. This latter method, however, is far superior to diagonal stirrups which are loose, as in tests of this kind, when the beam deflected, they dragged under the main bars, breaking away the concrete and causing failure at a lower load than a beam which had no web reinforcement whatever. Loose vertical stirrups gave better results than beams without any web members. It is harder work to embed diagonal web members properly, as shown on page 283, than with vertical ones, or where some of the main bars are bent diagonally upward to the top of the beam. Therefore, to obtain as good results with the inclined stirrups requires more expense and smaller stone in the concrete than is required with other types.

This discussion, however, can be made entirely independent of that of flexure, and final judgment should be reserved until more experimental data have been obtained. Some of the colleges are conducting such experiments at the present time, the results of which it is expected will throw considerable light upon this subject. The writer, therefore, has merely stated his experience, which may help toward the ultimate solution.

Mr. Goodrich. E. P. Goodrich, M. Am. Soc. C. E.—For several years the speaker has had in mind the use of a simple formula, such as the one suggested by Captain Sewell, to be used in the design of reinforced concrete beams. With this in view, the results of tests of several hundred beams have been carefully analyzed and plotted. After considerable work had been done, the speaker's attention was called to a paper by T. L. Condron, M. Am. Soc. C. E., entitled "A Study of Tests of Reinforced Concrete Beams," presented before the American Society for Testing Materials. In it Mr. Condron has analyzed eighty-three tests, and has proposed a formula of the form.

(Moment of forces) $\div$ (depth $\times$ area of beams) = (constant $\times$ percentage of steel) + (another constant).

The paper is most interesting and is worthy of close study.

Several engineers had been making use of simplified formulas before L. J. Johnson, M. Am. Soc. C. E., published, in *Engineering News*, his table of the values of K (which corresponds with Captain Sewell's h). As far as the writer is aware, however, Professor Johnson was the first to put the theoretical side of the subject into such shape that a simple formula was available for practical use.

Because of the diversity of composition and age of the tested

Mr. Goodrich.

beams, reports of which were available for examination by the speaker, it was difficult to determine the accuracy of Mr. Condron's deductions. The speaker, therefore, instituted the following test: Seven beams were built, with tension rods only, and all had the same area of steel and the same area of concrete above the reinforcement. The breadth and depth of the several beams were varied as far as possible. With these conditions, the breaking load should vary directly with the depth of the beam, according to Mr. Condron's equations. The beams broke with remarkable uniformity, all failing by crushing.

Fig. 21 shows the plotted results. The results also clearly show the possibility of using such a formula as that proposed by the author, at least when a single percentage of steel is used.

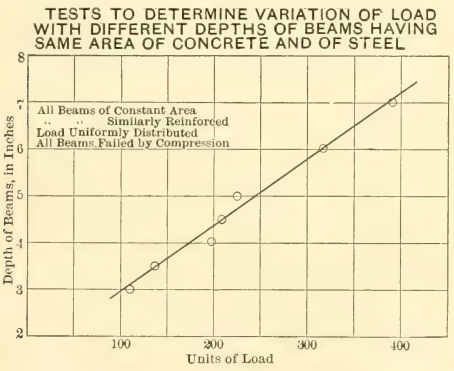


FIG. 21.

The analysis of the various test beams developed several different points, one of which was the fair constancy of a quantity corresponding with the h of the author's formula. Even the several hundred examples, however, were found to be altogether too few, upon which to base perfect reliance, and the speaker has thus been forced to resort largely to theory in order to develop working rules for design; but these rules are well tinctured with the deductions from the experiments analyzed.

The author's work, in analyzing the Watertown compression tests to arrive at a proper assumption as to the stress-strain curve, is in direct line with what he states must be done as a solid basis upon which to construct working formulas. It is an indirect method, but, nevertheless, of great value.

Mr. Goodrich. His objection, that the assumption of a right line as the stress-strain curve for concrete gives beams which are unnecessarily heavy, may be true from the point of view of theory only, and perhaps may be proved so in practice when only the best of workmanship is exacted. However, when consideration is given to the fact that probably a large majority of the reinforced concrete work going on at the present time is being done by inexperienced contractors, employing relatively low-grade labor, and under little or no supervision, the speaker believes that the Building Department of the City of New York, the Prussian Government, and the French Commission have taken a wise course in specifying the right-line assumption. In the example given by the author, he shows an increase of only ½ in. for a beam which would have a depth of 8 in., according to his method of design. This is an addition of only 6¼%, which is very small to cover even ordinary variations in workmanship, and is only 3¾% more than enough to cover the variation from the nominal weight of steel bars allowed in the rolling mills, according to standard steel specifications.

On the basis of concrete at 20 cents per cu. ft., steel at 3 cents per lb., and 1% of steel used as reinforcement, a variation in cost of only 3.6% is found between the right line and the author's assumptions. While bars usually run over weight, still there is a real chance of a possible deficiency, in the carrying power of a beam, of as much as 2½%, against which safe practice recommends a constant increased cost of only 3.6 per cent.

What the author says as to the proper definition of the modulus of elasticity of concrete is only too true. The speaker has been unable to find any data as to the modulus for concrete 14 days old, which is the age at which he determines all working values, and has been forced to assume one until such time as accurate tests can be secured. He has lately come to the conclusion that the usual ratios assumed for the moduli are too small.

Fig. 22 shows the curves representing the theoretical percentage of depth from the top of a beam to the neutral axis, for different percentages of steel and ratios of moduli, based on a right line for the stress-strain diagram. On it are also plotted the actual locations of the neutral axis determined by measurements on the beams tested by A. N. Talbot, M. Am. Soc. C. E., and reported to the Western Society of Engineers. It is seen that the curve coming nearest to the observed results would be for approximately 18, as a ratio of moduli.

Another illustration, tending to prove this conclusion, was of a test beam designed with stirrups of the special type adopted by the speaker. When only 14 days old it failed by compression under a center load of 16 000 lb. The effective beam area was 7 in. wide by

10 in. deep, and the clear span was 100 in. On the assumption of a right line for the stress-strain diagram, and a ratio of 12 for the ratio of moduli, the extreme fiber stress at failure was 3 800 lb. With a ratio of 20, the fiber stress was 2 750. This means one or both of two things: the ratio of moduli assumed in the first case was altogether too small, or the type of stirrup had much to do with raising the extreme fiber stress. Probably both are true. These points, with many others, make the speaker feel that values for r below 15 are really too small. This is most likely to be true with certain designs of web reinforcement which act to stiffen the beam. Mr. Goodrich.

The speaker agrees with the author when he says that the "most logical" method of design is to compute the load producing the

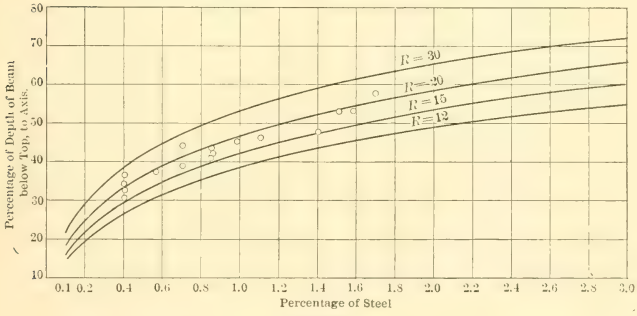

LOCATION OF NEUTRAL AXIS FOR TALBOT'S TESTS WITH REFERENCE TO THE THEORETICAL LOCATION FOR VARIOUS RATIOS OF THE MODULI OF ELASTICITY, USING A RIGHT LINE AS STRESS-STRAIN CURVE FOR CONCRETE
Percentage of Depth of Beam below Top, to Axis
R = 30
R = 20
R = 15
R = 12
10
20
30
40
50
60
70
80
0.1 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0
Percentage of Steel

FIG. 22.

maximum stress "as the dead load, plus the product of the working live load by a factor of safety." That is the method adopted by the speaker for all his work.

It is interesting to note that, after all his work, the author finds that the average values of his quantity, k, vary only from 0.865 to 0.84, even with the widest range of theory as to the form of the curve in question. When he assumes a constant value of 0.85, he strikes almost exactly the one found for a steel stress of 40 000 lb. per sq. in. Both 40 000 and a value very close to 0.85 are the ones adopted by the speaker for all his ordinary work. In selecting 40 000, he agrees with the author as to adopting the elastic limit of the steel as one maximum stress.

Mr. Goodrich.

The speaker is also in substantial agreement with him as to the propriety of adopting, as a working value for the maximum stress for concrete, an amount reduced appreciably below the ultimate stress usually assumed for that material. The author, like most engineers, adopts 2 500 as the ultimate stress. He, however, reduces this amount by 20% and uses 2 000 as the maximum allowable stress on concrete. The speaker uses this same figure, but arrives at it by a different course of reasoning.

It is believed that almost all structural work, both of steel shapes and of reinforced concrete, is likely to receive its most severe test during erection, and that (in the case of concrete at least) it is most apt to occur about 14 days after the concrete has been placed. The centers are then removed or are being removed, and the structure is in such a condition that building materials and all sorts of things are likely to be piled about in the promiscuous way so well known to all engineers. Little thought is ever given to the actual weight involved, and overloading is known to be only too common.

The diagram found on page 257 of Taylor and Thompson's book, "Concrete, Plain and Reinforced," shows that the average ultimate compression stress in concrete 14 days old is about 85% of its ultimate stress at the age of one month. This tends to show that the reduction of 20% advocated by the author is a wise one, especially when conditions are considered which are apt to occur at an early age of the concrete.

For these reasons the speaker has prepared all his working formulas on a basis of 2 000, which is lower than has generally been used by most engineers, except through the devices of a relatively low safe working stress for concrete, or of a larger factor of safety for concrete than for steel.

The author suggests that experiment is best to determine "the maximum allowable percentages of steel for each grade of concrete." In this connection, the deductions made by Mr. Condron are of interest. He says:

"For plain steel bars of approximately 33 000 lb. per sq. in. elastic limit.....P (is) not to exceed 1.5%," and for "corrugated or similar bars of approximately 55 000 lb. per sq. in. elastic limit.....1¼ per cent."

He does not give his reasons for the deductions, and the method of loading, and size and kind of reinforcement in the beam influence so largely the type of failure that some modification of such a general rule should be made, so as to meet varying conditions.

In this connection, the results of the tests made by H. A. Carson, M. Am. Soc. C. E., published in the report of the Boston Transit Commission for 1904, are of great interest. Fig. 23, at the left, shows the plotted results of tests of plain, square and other bars of

low elastic limit. In all cases the failures are described as being due to "tension," except two with small percentages, in which one bar slipped. Fig. 23, at the right, shows results obtained with twisted and corrugated bars. In the tests marked with double circles, the beam failed by what the report designates as "shear," but which the speaker believes to be due primarily to a splitting of the concrete along the line of reinforcement because of the shape of the bar and its action when it begins to draw through the concrete as tension comes upon the reinforcing steel. Mr. Goodrich.

Since the beams reinforced with plain rods did not fail by shear or by slipping of the rods, even when the percentage of reinforcement was high, there is nothing to lead one to presuppose failure by shear in the cases of the beams with twisted and corrugated bars.

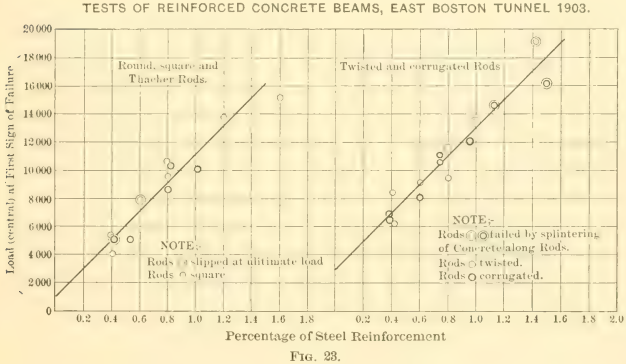


FIG. 23.

Fig. 1, Plate XXV, is a reproduction of one of several photographs contained in the report, to which reference is specifically made as illustrating the type of failure in these cases. It may be contended that the bars were spaced too closely, so that not enough concrete surrounded the bars to bring them into proper action. This is doubtless true, but when sufficient concrete is available for this purpose, the design is a failure from the economical standpoint.

The tests made by Mr. Carson show that—for beams of the size involved, reinforced simply by tension bars, and broken by a center load—not more than 1% of any but plain steel bars should be used. It will be seen, later, that when stirrups are used, much larger percentages of steel can safely be utilized.

Mr. Goodrich. The analytical work carried out by the author is extremely interesting in itself, but it is even more so when it is examined as to its agreement or disagreement with actual tests.

The speaker has investigated a great many theoretical designs based on almost every combination of conditions, and one set of results is shown in Fig. 24. The depths of beams of constant width, designed to support a given total load on a given span, with varying percentages of reinforcement, were computed. One curve was plotted showing the varying depth when only the steel was considered. Another curve was then found when the concrete alone

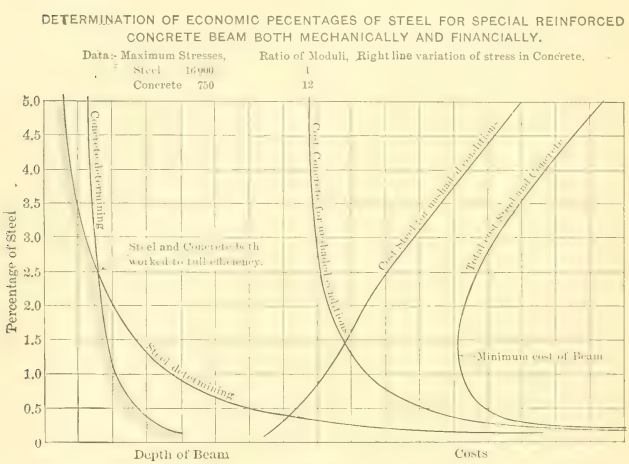


FIG. 24.

was kept in view. These two curves are at the left in Fig. 24, and where they cross is the only point at which both materials are worked to their full mechanical efficiency.

With a percentage of steel other than that found at the point of intersection of the curves, the depth is determined by one or other of the unshaded parts of the lines. The costs of both steel and concrete for beams of the depths determined by these unshaded lines were then computed, and are shown in the next two intersecting curves. The total cost is given by the line at the right, which shows a decided minimum. In this special case, however, the cost varies only slightly between 1 and 1½% of steel.

Mr. Goodrich.

With other assumptions as to the ratio of moduli, of the maximum allowable stresses, and of the unit costs of the materials, the minimum occurs with other percentages of steel.

Some actual plotted costs are shown in Fig. 25, which includes Professor Talbot's tests with corrugated bars, and the Boston tests with square and with twisted bars. The minimum occurs between 1 and 1½% of reinforcement for the special prices assumed, *viz.:* 20 cents per cu. ft. for concrete and 3 cents per lb. for steel. These are the cost prices assumed by the author.

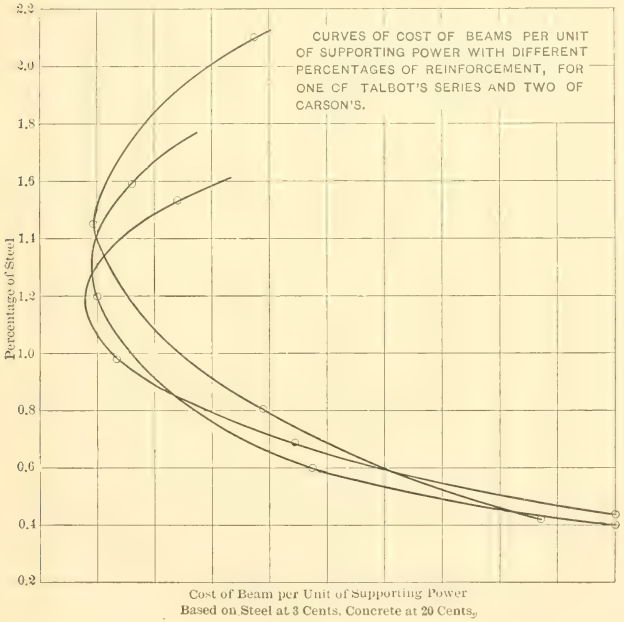


FIG. 25.

With respect to possible economies which might be effected by introducing steel in the compression edge of the beams without using it in the web also, only the tests for the Boston Tunnel shed any light, as far as the speaker is aware. However, they entirely bear out the author's deduction, that "single reinforcement" will be

Mr. Goodrich. cheaper. The left-hand portion of Fig. 26 shows the results obtained, and little increase in supporting power is observable, probably because all beams will fail, by so-called "shear" in most cases, long before the top reinforcement rods can be brought into action, unless special means are provided to make them do so.

From some experiments on the longitudinal reinforcement of columns which the speaker has seen, he feels certain that, in both beams and columns, steel designed to take compression, under such conditions, carries but a small percentage of what it is usually calculated to do. If it is necessary to increase the compression side of a girder with steel, it can be done much more economically in other ways. The beam described earlier, which broke with an extreme fiber stress of more than 2 750 lb. per sq. in., tends to prove this.

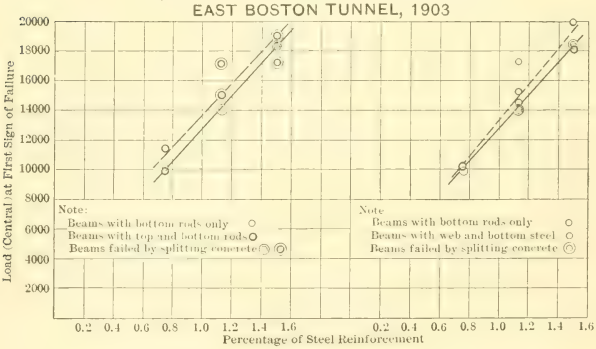


FIG. 26.

The author's paragraph with regard to the ideal web reinforcement should be fruitful of interesting discussion. Personally, the speaker does not agree with most of the clauses contained in it. However, he believes thoroughly in web reinforcement, and that it should consist of a multiplicity of small members which should extend to the top of the beam. The experiments made for the Chicago, Milwaukee and St. Paul Railway by Mr. J. J. Harding, and reported to the Western Society of Engineers, strongly tend to prove each of these items, but especially the first and last; while other experiments witnessed by the speaker some years ago led him to the conclusion contained in the second point, to which he called attention in his discussion of Captain Sewell's paper* before the International En-

*Transactions, Am. Soc. C. E., Vol. LIV, Part E, p. 459.

gineering Congress. In the other points, issue is taken with this paper; however, these matters are not necessary to the analytical treatment, the results of which are believed by the speaker to be essentially correct and to be borne out by experiment. Mr. Goodrich.

In exact accord with the author's ideas, the speaker designs beams primarily with regard to the maximum moments, without reference to anything except an approximate dead load. The latter is usually taken as 1 lb. per sq. in. of "guessed-at" cross-section per foot of length. Afterward, web reinforcement is added, and all the parts thus determined are investigated as to other possibilities of failure.

The best comparative experiments known to the speaker, illustrating the value of web reinforcement, are those made by Mr. Harding, to which reference has been made. He found that his web reinforcement increased the carrying power of the beams which contained it about 50%, and that the variation in strength was much less in beams with web reinforcement than in those without it. Mr. Harding experimented with only a single percentage of steel. In some tests for the Boston Tunnel, a wide variation in the percentage of tension reinforcement was made. The same relative effect of web reinforcement was observed, but no results as striking as those of Mr. Harding were obtained. In the curves shown at the right of Fig. 26, beams with bottom rods without web steel are compared with those with bottom rods and with vertical web reinforcement. The value of the latter will never be apparent until the load has reached a point beyond which the concrete cannot withstand the developed stresses. In Professor Talbot's discussion of Mr. Harding's paper, this value is given as:

Vertical shear $\div$ (breadth $\times$ effective depth) = from 125 to 150 lb. per sq. in.

It is well known that when conditions are right, concrete can develop a very large shearing resistance, and the speaker believes that properly designed beams should take account of this capacity. In this point he differs from the author, who states that "ideal web reinforcement" should be of such size that "the sum of the horizontal components of the stresses in all the web members in each half of the beam should be at least equal to the maximum stress in the flange reinforcement." The speaker believes that this would give an excessive amount of web steel.

Turning to the Boston experiments, it is seen that the beams provided with web steel show little improvement over beams without it, until a point is reached above which the latter beams fail by "shear"; but, even then, only small improvement is found, due probably both to the design of the web reinforcement and the relative depths of the beams.

Mr. Goodrich. When comparison is made between beams which have both top and bottom rods, with and without web reinforcement, or beams which have web reinforcement with and without top rods, a decided improvement is apparent. The results of such comparisons are shown in Fig. 27, in which, except where stated to the contrary, loadings are given at the first sign of failure. When ultimate loads are considered (at which the speaker usually looks askance), a still more striking improvement is apparent. It should also be noted

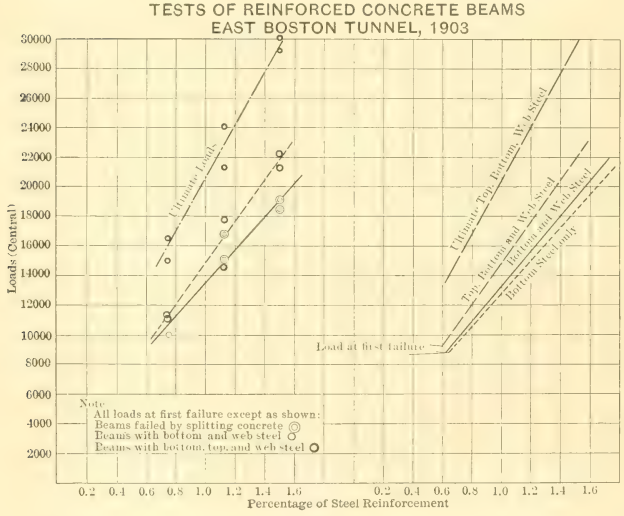


FIG. 27.

that the results given for Mr. Harding's experiments refer to loads at ultimate failure.

Unless a large percentage of tension steel is used, and the beams are comparatively deep, it will not usually be found financially profitable, purely from the point of view of increased safe load capacity, to use web reinforcement; and, even with large percentages of tension steel, it is rarely profitable unless the increase in the ultimate loads is noted, and advantage is taken to reduce the required factor of safety accordingly.

But, laying aside all ideas of economy, the speaker believes emphatically in the use of ample web reinforcement, because of the

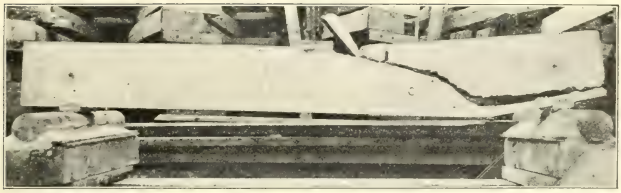


FIG. 1.—BEAM TESTED FOR BOSTON TUNNEL, WITHOUT STIRRUPS, SHOWING TYPE OF FAILURE.

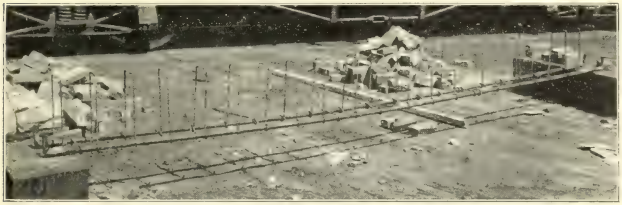


FIG. 2.—REINFORCEMENT USED IN BRICK BEAM TEST.

FIG. 3.—BRICK BEAM, NOT LOADED.

FIG. 4.—BRICK BEAM SUPPORTING LOAD.

marked difference in the type of failure which will take place. Mr. Goodrich. Fig. 1, Plate XXVI, shows a beam from the Boston series which was reinforced with bottom and web steel and carried more than 19 000 lb. at the first sign of failure. Even when ample factors of safety are used, it is well worth the cost of web reinforcement to be sure that when failure does occur it will be like Fig. 1, Plate XXVI, rather than like Fig. 1, Plate XXV.

It was from a study of such conditions as are shown in the case of top rods with web steel that the speaker was led to adopt, for his work, a type of web reinforcement which did not depend for its action on its adhesion to the concrete. That is the only means that most systems have of transmitting stresses from the web steel into the concrete, whether the web steel takes the form of straight spines, or consists of vertical or other shaped members primarily designed as aids to easy construction. A much better form, in the speaker's opinion, is one in which the web steel is actually bound to the floor slab reinforcement. In this case, however, the latter should run across the top of the beam. But still better for construction purposes is a design in which the tension rods are placed in pairs, and the web steel is shaped like an inverted U with the free ends wrapped around the tension rods. The speaker claims no originality for this design, although he believes he was among the first to see clearly its mechanical and constructional advantages, and first worked out its proper design. By this system, the web steel may or may not be rigidly connected to the tension rods. In some beams, rigid connection is well, especially where the reinforcement is fabricated at a point distant from the point of installation, and the amount of handling during shipment is large.

The speaker must include himself among those to whom the author refers, and who cannot agree with him that "attached web members are necessary." The speaker believes that the two following tests, devised and carried out under his direction, go far toward disproving the need of the rigid attachment of web members.

A "truss" was built, consisting of tension bars hooked at the ends and of inverted U-shaped stirrups with their ends simply wrapped around the tension rods. The stirrups were spaced so that ordinary hard building brick could be placed between them. Flat plates were set in front of the bent ends of the tension rods to prevent the end bricks from being cut by the rods, and oak spacing pieces, of just the thickness of the stirrups, were used to separate the bricks which would otherwise rest against the stirrups and be cut by them. Figs. 2, 3 and 4, Plate XXV, show the plain reinforcement, the brick beam, and the load carried when the top bricks began to crush, respectively.

A more interesting experiment lately carried out by the speaker was the following:

Mr. Goodrich. A concrete beam was moulded which was reinforced with a truss similar to that used in the brick beam. However, the concrete was not allowed to get below the bottom rods except at the points of support. In this way no possibility of any adhesion between the tension rods and the concrete existed. Furthermore, there was no rigid connection between the bottom rods and the stirrups, as the latter were simply wrapped around the former. At an age of 14 days the beam failed by crushing at the center, with a load of 9 326 lb., while a similar beam, built without stirrups, failed under a center load of 9 000 lb. by rods pulling out of the concrete. Fig. 2, Plate XXVI, shows the latter beam, and Fig. 3, Plate XXVI, shows the failure of the beam which had stirrups, but had no concrete under the tension rods. Fig. 4, Plate XXVI, shows the beam tipped upside down after it had been broken, so as to show the method by which the stirrups were attached to the tension rods.

The design of web reinforcement adopted by the speaker also materially increases the resistance which the beam will develop to compression, as it acts in a manner similar to that of the hooping or spiral steel used to reinforce columns. Moreover, the increased crushing strength is developed in both beams and columns, with only a fraction of what is required where longitudinal rods are used, as the tests of Considère tend to prove.

The test described earlier in this discussion, in which was developed an extreme fiber stress of from 2 750 to 3 700 lb. per sq. in. for concrete 14 days old, is a strong proof of the advantage claimed.

Whenever possible, of course, recourse is had to the device of using the floor structure adjoining the compressed edge of beams to increase the compression area of the beam, but the speaker has never been satisfied as to the accuracy of the methods of computation thus far published for such beams. It is something of a disappointment that the author did not give some analysis of the relations which should exist between his quantities, s, b and c, in T-beams. He takes for granted a predetermined relation which is not involved in the determinations of economic dimensions, and perhaps he is right. In this point, it is concluded that he uses the same methods of determining the quantities in question as he gave in his paper before the International Engineering Congress, in which he followed A. L. Johnson, M. Am. Soc. C. E. It is a great pity that so few experimenters have tested beams of T-section, because they are the ones oftenest encountered, and every designer makes large use of the flanges of the T in his designs.

The analysis of the several hundred beams examined by the speaker has convinced him that reinforced concrete beams designed with vertical stirrups like those shown in Fig. 2, Plate XXV, may

FIG. 1.—BEAM TESTED FOR BOSTON TUNNEL, WITH STIRRUPS AND BOTTOM RODS, SHOWING TYPE OF FAILURE.

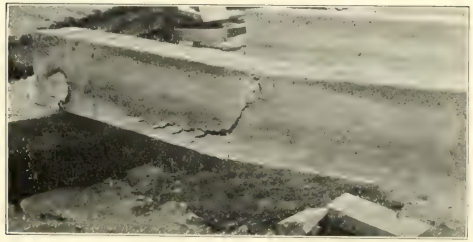


FIG. 2.—BEAM, 14 DAYS OLD, TESTED BY WRITER. FAILED BY PULLING OUT OF HOOK. NO STIRRUPS USED.

FIG. 3.—BEAM, 14 DAYS OLD, TESTED BY WRITER. FAILED BY COMPRESSION. NO CONCRETE AROUND TENSION RODS. STIRRUPS USED.

FIG. 4.—SAME BEAM AS FIG. 3, TIPPED UPSIDE DOWN TO SHOW ATTACHMENT OF STIRRUPS, AND ABSENCE OF ALL ADHESION.

be analyzed like a multiple-system Howe truss, as far as most points in the design go. He is borne out in this idea by three tests of full-sized members which were executed during the past year in connection with the construction work going on under his direction. Mr. Goodrich.

The floors for one building were designed to carry a load of 800 lb. per sq. ft. at the first crack, when 30 days old. Two full-sized modified T-beams were broken at an age of 14 days. They were designed for the building work as partially anchored at the ends, so that the denominator in the moment formula was 10 instead of 8, as for beams simply supported at the ends, and as the tests necessarily must be. Rails were loaded upon the test beams, and their weight was sufficient to cause failure without the possibility of arching or of other troublesome effects. The actual load safely carried by the tests when the first cracks appeared, when increased in the ratio of 10 to 8, was equivalent to a load of 815 lb. per sq. ft.

Four months after one floor of the building had been completed, four full bays, aggregating a total area of about 1 000 sq. ft., were loaded with brick to aggregate 875 lb. per sq. ft. Under this load, a fine crack appeared at the center of the bottom of the two fully-loaded girders, with no signs of strain visible at any other point except small deflections. Great care was taken to pile the brick so as to prevent any arching effect, and, as the greatest deflections were slightly more than $\frac{1}{4}$ in. for 17-ft. spans, little reduction of load from arching could have occurred in any case.

The third test was of a beam designed to carry a central load, and illustrates two important points, in spite of the fact that the concrete was of very inferior quality, for some reason as yet unfathomed by those in charge. Figs. 1 and 2, Plate XXVII, show the beam as a whole, and a near view of the principal failure. Just as the last rail was applied to the test, tension cracks appeared near the center of the beam. At almost the same time, spalling of the concrete took place at the top, at the center. This might have been hastened by the action of the load, but only slightly so, as good supports were provided to distribute the load properly. Before another rail could be added, the failure shown in the photograph took place. It is one in which a whole panel between two of the vertical stirrups seemed to shear out (using the word in its proper sense). There was no indication of failure by diagonal tension, according to the usually accepted idea of shear; and the failure cannot be attributed to horizontal shear, as the structure of the exposed concrete shows that the stresses were solely vertical ones. In this connection, it should be noted that most designers give the horizontal steel its full value in resisting vertical shear. It is believed that this method of design is proved to be of doubtful value by this experiment.

One point, clearly brought out, is that a type of failure, which

Mr. Goodrich. has not generally been considered, is possible when numerous stirrups are used. There was no pulling out of the bottom rod from the concrete at the point of support, and there was perfect action between the concrete and the unattached stirrups in this case. The failure occurred in the first panel away from the abutment, and at a load far less than the longitudinal steel would carry theoretically in shear.

One special point, from which the speaker draws some consolation with regard to this test, is that the quantities assumed in making up the design were so well balanced that the three principal types of failure showed themselves almost simultaneously. This shows that all parts of the structure were consistently proportioned and that, with safe loads, all parts would have equal factors of safety. Of course, with better concrete, this relation would not be quite so close, but then failures would have taken place through tension, and the other parts would have been very near failure as well. Of all types, a failure by tension is least to be feared, as the author intimates. However, only in beams with very low percentages of reinforcement will failure occur without a considerable crushing occurring after the steel has passed its elastic limit.

The speaker believes that the type of reinforcement he has adopted is far cheaper than that advocated by the author, and will be even more efficient in some cases. The experience of the past year, during which several thousand tons of such reinforcement were fabricated and worked into place under the speaker's direction, has shown that steel fabricated and ready for placing costs only about $45 per ton, and that the cost of placing in the forms is less than $\frac{1}{4}$ cent per lb.

When the author discusses the advantages of rigid attachment of web members, in regard to fire tests, he makes a statement about "web members wrapped around the tension bars" being inadequate. To this the speaker takes exception, and he feels that the test of the concrete beam illustrated in Figs. 3 and 4, Plate XXVI, tends to prove the fallacy of the author's statement.

The speaker thoroughly agrees with the author in all that he says in his conclusion about the advantages of having web reinforcement run quite to the tops of beams, but he would substitute the word "vertical" for "diagonal" in each case.

The speaker has been intensely interested in this subject for a long time, but feels that there is more experimental work available for analysis than the author admits in his last paragraph.

This opportunity is taken to give expression to a gradually growing feeling, on the speaker's part, that much of the analytical work now in vogue is based on wrong primary assumptions. At least one writer of a book on the subject of reinforced concrete has hinted at

FIG. 1.—BEAM WHICH FAILED BY TENSION, COMPRESSION AND END SHEAR, SIMULTANEOUSLY.

FIG. 2.—VIEW SHOWING CHARACTER OF FAILURE DUE TO END SHEAR.

the same thing, and an engineer of some eminence has lately broached the same idea in a paragraph of a public address which has been printed in an engineering periodical. Mr. Goodrich.

In any case, the thanks of the profession are certainly due to Captain Sewell for his timely and scholarly paper.

EDWIN THACHER, M. AM. SOC. C. E. (by letter).—The writer has no doubt that the author's Equation 0 will give as reliable results as any other formula, however complicated, provided the values, h and A, are well established. Mr. Thacher.

As noted by the author: "A must be expressed as some fraction of the area of concrete." A should bear such relation to d that the strength of the concrete in compression is equal to the strength of the steel in tension. If, in Equation 0, the values of A and T be substituted, it becomes $M = h\ d^2$, which is simpler still, and this is the formula giving the moment of resistance of a rectangular concrete-steel beam which has been used quite extensively by the writer and others for the past six years. It was first published in the *Transactions* of the Association of Civil Engineers of Cornell University,* and, since then, has been published in *Cement*,† in *Engineering News*,‡ and in a pamphlet issued by the Concrete-Steel Engineering Company, and is quite well known. In the same publications, formulas are given for the ultimate strength of beams, supported at the ends and loaded at the center, in terms of $\frac{d^2}{l}$; for beams uniformly loaded, in terms of $\frac{d^2}{l^2}$; and for finding the depth of beam to sustain any required load, l, being known. Nothing simpler is possible or could be desired. They are in the same form, and require no more labor in calculation than formulas for wooden beams.

In the writer's formulas, the stress-strain line is considered as straight, but, if considered as a parabola or any other curve, the only change in the formulas will be in the value of the constant coefficient, and if this is established by experiment it does not matter what shape of stress-strain curve is usual in calculation.

In the formulas above noted the constant coefficients were determined theoretically. The ultimate strengths of the concrete and steel were used; also such values as E_c as resulted from the highest pressures recorded. From the results of such tests as the writer has been able to work up, he has had no occasion to change the values of the coefficients. Five sets of tests, made in different localities, and by different men, cover a variation in composition of concrete from 1:2:4 to 1:3:6, a variation in age of specimens from 23 to 90 days, a variation in ratio of length to depth of 6.0 to 22.2, a variation in

* Vol. X, 1902.

† July, 1902.

‡ February 12th, 1903.

Mr. Thacher. strength of metal from 50 000 to 100 000 lb. per sq. in., and a variation in percentage of metal from 0.31 to 3.9 per cent. The maximum variation between actual and estimated strength in no case exceeded 16 per cent. The mean variation for any one set of tests in no case exceeded 2.3%, and the mean variation for all tests, 30 in number, was 0.013%, or practically zero.

In 9 of the 30 tests the reinforcing bars broke. The writer does not see how any formula for ultimate strength of concrete-steel beams can be even approximately true when the elastic limit of the steel is used in the formula, unless the constant coefficient is modified to compensate therefor.

The elastic limit of steel is about six-tenths of its ultimate strength. Considère and Professor Bach state that the elastic limit of concrete, as far as concrete can have an elastic limit, is also about six-tenths of its ultimate strength in compression, so that it appears to the writer that if the author uses the elastic limit of steel in his formulas he should use 60% instead of 80% of the crushing strength of the concrete. The writer agrees with the author that it would be very desirable to make numerous tests of beams using, say, 1:2:4 and 1:3:6 concrete, and containing various percentages of steel reinforcement, for the special purpose of establishing the values of h in his Equation 0, or the coefficient of d^2 in the writer's formula. From the result of Professor Hatt's experiments, the writer drew the conclusion that if the amount of reinforcement is double what it should be to equal the strength of the concrete the gain in strength of beam due to lowering the position of the neutral axis will be about 20%, and if the value of E_c is doubled there will be a loss of strength of about 20 per cent. The writer agrees with the author that the safe loads should be determined by a factor of safety, for the breaking loads can be found by tests, and, for any intermediate loads, the value of E_c is constantly changing. He does not agree with him, however, regarding the factor of safety recommended, that is to say, nothing for dead load and 4 for live load. If there is anything that will ultimately bring concrete-steel construction into disrepute, the writer believes it will be due to the reckless method of proportioning followed by some constructors. The writer never heard of a bridge or an arch being designed with a factor of safety of 1 for dead load, and the engineer who would undertake it would probably not repeat the experiment. The writer believes that if cost prohibits a factor of safety of at least 3 for both dead and live loads it will be better to use wood or some other cheap but safe construction. The investigations and formulas of the author regarding minimum cost are interesting and instructive, but the writer doubts whether they will find much practical application, as the depth of beams is frequently governed by practical considera-

tions, and the minimum percentage of steel is fixed by calculation, and any greater amount than that is partially wasted. The writer does not agree with the author that the attachment of the web reinforcement to the horizontal reinforcement should necessarily be independent of the concrete. As long as the web reinforcement is attached to the flange reinforcement, and is securely clamped thereto by the concrete, no movement can take place, and all conditions are satisfied. Mr. Thacher.

If, as the author considers, it is necessary to provide steel for all shearing stresses, it appears to the writer that all superfluous parts of the concrete should be removed, resulting in what is known as the Visintini system, which the writer considers a very excellent system, it being the application of the truss principle to concrete-steel construction.

H. T. Forchhammer, Assoc. M. Am. Soc. C. E.—Without doubt, reinforced concrete will be used instead of steel for many and various constructions, in the near future, and any contribution to the theory or practical design is warmly welcomed. Mr. Forchhammer.

The speaker thinks that Captain Sewell has done right in placing the economical design in the foreground, but does not quite approve the way in which the problem has been solved.

The stress-strain curve, as advised by the author, is probably the best one available until the results of experiments—made especially for this purpose—are at hand.

In opposition to the author, the writer believes that total failure occurs when the stress in the steel reaches the elastic limit. Hence, the ultimate strength of the beam is reached when the steel is stressed to the elastic limit, or the concrete is compressed to its ultimate strength.

The considerations for economical design, which will be given herewith, will show that it is not always the cheapest beam which is designed for these two values simultaneously.

The author advises the use of Equation 6 instead of Equations 1 to 5. If, in Equation 6, a is replaced by the proportion of d given on page 261 this equation will give practically the same results as Equations 1 to 5, but, if a and d are considered as variables, as on page 264 the connection between Equations 6 or 7 and Equations 1 to 5 is quite changed.

Suppose Equation 7 to be solved for d, using a value for a which is smaller than the one given on page 261, the result is a beam in which the concrete is stressed less than 2 000 lb. per sq. in. when the steel is stressed to its elastic limit. If—on the other hand—Equation 7 is solved for d, using for a a value larger than given on page 261, the result is a beam in which the concrete is

Mr. Forchhammer.

stressed more than 2 000 lb. per sq. in. when the steel is stressed to its elastic limit.

The result of the author's economical considerations, therefore, is that, for concrete beams with reinforcement of low elastic limit, it is cheaper to proportion the beam with less steel than that given on page 261, but the author says nothing about the relative cost of beams reinforced with various kinds of steel. This can very well be done, and the speaker disagrees absolutely with the author when, on page 263, he says it is impossible—with Equations 1 to 5—to express the cost in terms of the ratio of cost of steel per cubic foot to the cost of concrete per cubic foot, because this ratio is entirely independent of the ratio between the maximum allowable stresses of the two materials. Just because these two ratios are entirely independent of each other it is possible to find a value for the last-named ratio, leading to minimum cost. To accomplish this the speaker will use Equations 1 to 4, and, instead of Equation 5, will use Equation 6.

With $b = 1$, $\frac{E_s}{E_c} = e$.

Let $F =$ the maximum compression in concrete.

Using Equations 3 and 1:

$$\frac{y_1}{1} = \frac{y_2}{\frac{f_s}{e\ F}} = \frac{d}{1 + \frac{f_s}{e\ F}}.$$

Substituting β for $1 + \frac{f_s}{e\ F}$, and solving:

$$y_1 = \frac{d}{\beta},$$

$$y_2 = d\left(1 - \frac{1}{\beta}\right).$$

This, substituted in Equation 4, gives

$$a\ f_s = 0.57\ F \times \frac{d}{\beta}, \text{ which, substituted in Equation 6, gives}$$

$$m = h \times d \times 0.57\ F \times \frac{d}{\beta}.$$

Substituting α for $\frac{0.57\ F\ h}{\beta}$, and solving:

$$d = \sqrt{\frac{m}{\alpha}}.$$

To express h by f_s, Fig. 1 gives

$$h\ d = y_2 + n\ y_1,$$

$$h = 1 - \frac{1 - n}{\beta}.$$

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for $$a = 0.64, \quad h = 1 - \frac{0.36}{\beta}.$$

Substituting the value of a in Equation 9,

$$x = p \times 0.57\ F \times \frac{d}{\beta\, t_s} + d,$$

as $$0.57\ F \times \frac{h}{\beta} = a,$$

and $$d = \frac{m}{\sqrt{a}},$$

$$\frac{x}{\sqrt{m}} = \frac{1}{\sqrt{a}} + \frac{\sqrt{a}\, p}{h\, t_s},$$

as $$\beta = 1 + \frac{t_s}{e\, F},$$

$$h = 1 - \frac{0.36}{\beta}$$

$$a = 0.57\ F \times \frac{h}{\beta},$$

it will be seen that x is expressed in terms of t_s, e, F, and p.

As e and p are constants, x is a function of F and t_s only; hence, for each value of t_s, there is a certain value of F, making x a minimum, or, for each value of F, there is a certain value of t_s, making x a minimum.

These values, naturally, are purely theoretical, and may be outside the practical existing limits; but as the special object is not to get an absolute minimum, but only a good economical design, the speaker is of the opinion that it may be of interest to find values for the relative cost of beams designed for various values of t_s and F.

In the formula just developed,

$$\frac{x}{\sqrt{m}} = \frac{1}{\sqrt{a}} + \frac{\sqrt{a}\, p}{h\, t_s},$$

the first member, $\frac{1}{\sqrt{a}} = \frac{d}{\sqrt{m}}$, is due to the cost of the concrete; the last member, $\frac{\sqrt{a}\, p}{h\, t_s}$, is due to the cost of the steel. In Tables 3 to 7, C has been substituted for $\frac{1}{\sqrt{a}}$, and S for $\frac{p\sqrt{a}}{h\, t_s}$. In these tables, in accordance with the author's opinion, p is assumed to be 72, and e to be 15. The ultimate strength of concrete in compression is assumed to be 2 300 lb. per sq. in. The upper part of Table 3 is for medium steel, with an elastic limit of 33 000 lb. per sq. in. For the various percentages of steel, the speaker has calculated the compres-

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sion in the concrete, represented by F; the depth of the beam, by $\frac{d}{\sqrt{m}}$; the cost of the concrete, by C; the cost of the steel, by S; the total cost, by $\frac{x}{\sqrt{m}}$; and the resisting moment is for a beam 1 in. wide and 12 in. deep.

It will be seen that the cheapest beams are those in which the percentage of steel is from 1.1 to 1.4, and that the cost does not vary much if the percentage is only a little outside of these limits.

The lower part of Table 3 is for hard steel, with an elastic limit of 57 000 lb. per sq. in. For the other items the values are the same as in the upper part of Table 3.

It will be seen that the cheapest beam is that in which the full strength of the concrete and the elastic limit of the steel are reached simultaneously. Here, also, a little variation in the percentage of the steel does not influence the cost very much; but, as there is no actual minimum value (but two independent curves intersecting at the minimum point), it is here more important to keep close to the right percentage.

To try these results on tests the speaker used a very interesting series of tests* made under the supervision of A. N. Talbot, M. Am. Soc. C. E.

The upper part of Table 4 is for 12-in. beams of medium steel (all in the series) with an elastic limit varying from 30 000 to 35 000 lb. per sq. in. This table shows the actual percentage of steel; the final stress in the steel; the breaking moment, in inch-pounds per inch; and the cost,

$$\frac{x}{\sqrt{m}} = \frac{12}{\sqrt{m}}\left(1 + 0.72 \times \frac{100\,a}{d}\right).$$

From Table 3 is taken the breaking moment and cost, as calculated, assuming the elastic limit to be 33 000 lb. per sq. in.

With one exception (Beam No. 9) it will be seen that the final stress in the steel varies between the limits 29 300 and 37 400, with an average of 32 900 lb. per sq. in. With the same exception there is also harmony between the ultimate moments and costs as they were in the actual beams and as they are calculated. It would seem that the calculated moments are somewhat too small for the small percentages and somewhat too large for the large percentages.

The lower part of Table 4 is for six beams of hard steel having an elastic limit of from 55 000 to 60 000 lb. per sq. in. In this table, also, there is evidence of harmony between theory and practice.

To complete the investigation, Table 5 shows the relative cost of beams reinforced with steel of various elastic limits.

*The results of these tests are published in Bulletin No. 1 of the University of Illinois Engineering Experiment Station, September 1st, 1904.

TABLE 3.—COST OF REINFORCED CONCRETE BEAMS, WITH VARIOUS PERCENTAGES OF REINFORCEMENT. Mr. Forchhammer.

MEDIUM STEEL.

$100\frac{a}{d}$.	$ts.$	$F.$	$\beta.$	$h.$	$a.$	Cost Data. $C = \frac{d}{\sqrt{m}}$	$S.$	$\frac{x}{m}$	For $d = 12$ in., $m.$	Remarks.
0.41	33 000	850	3.59	0.900	122	0.0906	0.0267	0.1173	17 600	
0.52	33 000	980	3.25	0.889	153	0.0809	0.0303	0.1112	22 000	
0.83	32 000	1 300	2.70	0.867	238	0.0649	0.0388	0.1037	34 200	
1.11	33 000	1 560	2.42	0.851	312	0.0567	0.0455	0.1022	44 900	
1.39	33 000	1 800	2.23	0.838	386	0.0510	0.0510	0.1020	55 300	Minimum cost.
1.56	33 000	1 940	2.14	0.832	429	0.0488	0.0543	0.1026	61 700	
2.03	33 000	2 300	1.96	0.816	546	0.0429	0.0625	0.1054	78 600	

HARD STEEL.

0.42	57 000	1 400	3.55	0.899	216	0.0681	0.0206	0.0887	31 000	
0.70	57 000	2 020	2.88	0.875	349	0.0536	0.0270	0.0806	50 200	
0.87	57 000	2 300	2.65	0.864	427	0.0484	0.0302	0.0786	61 500	Minimum cost.
0.97	53 200	2 300	2.54	0.858	442	0.0476	0.0332	0.0808	63 800	
1.52	39 800	2 300	2.16	0.833	505	0.0445	0.0486	0.0931	72 600	

Assuming the ultimate strength of concrete in compression to be 2 300 lb. per sq. in., it will be seen that the cost decreases with the elastic limit of the steel until this has reached about 100 000 lb. per sq. in. Any further increase in the elastic limit will not decrease the cost.

Naturally, this value for the elastic limit is purely theoretical, but the interesting point is, that with steel having an elastic limit of about 60 000 lb. per sq. in., practically the minimum of cost is reached, as the theoretical minimum is only 8% lower. (It must be remembered that this percentage is only part of the cost.)

Table 5 also shows that the saving effected by using hard instead of medium steel is but slight, compared with what it would be in a steel construction. Using steel with an elastic limit of 35 000 instead of 55 000 lb. per sq. in., the cost here considered will be increased by about 24%; and, assuming the part of the cost here considered to be about one-half of the total cost, it is only 12% cheaper to use the hard steel.

As medium steel is considered much safer than hard steel, for all structural purposes, especially where it is subject to impact, the speaker thinks it proper to consider the question whether the slight saving in cost really justifies the use of hard steel.

Another advantage in using medium steel is that the concrete is stressed less, hence the working stresses in the tension and also in

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TABLE 4.—Cost of 12-in. Reinforced Concrete Beams with Various Percentages of Reinforcement.

Medium Steel.

Beam No.	Reinforcement in 12 × 12-in. beam.	Percentage of Steel.	Final stress in steel measured.	Breaking Moment, m, per 1 in. width of beam.		Total Cost. $\frac{x}{\sqrt{m}}$	
				Actual.	Calculated from Table 3.	Actual.	From Table 3.
19	3 ½-in. plain round.	0.41	37 400	21 400	17 600	0.106	0.117
21	3 ½-in. " "	0.41	32 900	18 700	17 600	0.113	0.117
9	3 ½-in. Ransome	0.52	70 000	42 000	22 000	0.081	0.111
16	3 ½-in. plain square.....	0.52	32 100	23 100	22 000	0.108	0.111
17	3 ½-in. " "	0.52	29 300	22 200	22 000	0.110	0.111
5	3 ½-in. Kahn.....	0.83	30 600	30 400	34 200	0.110	0.104
10	3 ½-in Thacher.....	0.83	32 000	33 800	34 200	0.104	0.104
15	3 ½-in. "	0.83	35 000	35 200	34 200	0.101	0.104
14	4 ½-in. Kahn.....	1.11	30 200	39 700	44 900	0.108	0.102
4	5 ½-in "	1.39	33 800	49 000	55 300	0.108	0.102
27	4 ½-in. plain square.....	1.56	24 900	58 300	61 700	0.105	0.103
22	3 ¼-in. Kahn.....	1.67	33 800	51 400	66 000	0.115	0.103

Hard Steel.

3	3 ½ in. Johnson.....	0.42	52 600	28 000	31 000	0.095	0.089
7	3 ½-in. "	0.42	58 300	30 300	31 000	0.091	0.089
2	5 ½-in. "	0.70	58 800	44 300	50 200	0.086	0.081
20	5 ½-in. "	0.70	65 200	46 600	50 200	0.083	0.081
13	7 ½-in. "	0.97	54 100	64 100	63 800	0.080	0.081
28	6 ½ in. "	1.52	50 300	72 400	72 600	0.093	0.093

TABLE 5.—Cost of Beams Reinforced with Steel of Various Elastic Limits.

f_s.	F.	$100\frac{a}{d}$.	β.	h.	α.	Cost-Data. $C=\frac{d}{\sqrt{m}}$.	S.	$\frac{x}{\sqrt{m}}$.	For $d=12$ in., m.	Remarks.
30000	1 630	1.39	2.23	0.838	348	0.0536	0.0586	0.1072	50 100	
33 000	1 800	1.39	2.23	0.838	386	0.0510	0.0510	0.1020	55 300	
35 000	1 910	1.39	2.23	0.838	407	0.0496	0.0496	0.0992	58 600	
42 300	2 300	1.39	2.23	0.838	493	0.0451	0.0451	0.0902	71 000	Minimum depth.
50 000	2 300	1.07	2.15	0.853	456	0.0469	0.0361	0.0830	65 700	
55 000	2 300	0.92	2.59	0.861	435	0.0480	0.0818	0.0798	62 700	
60 000	2 300	0.80	2.74	0.869	415	0.0491	0.0288	0.0774	59 700	
70 000	2 300	0.62	3.03	0.881	381	0.0513	0.0229	0.0742	55 000	
80 000	2 300	0.49	3.32	0.892	352	0.0534	0.0188	0.0722	50 600	
90 000	2 300	0.40	3.61	0.900	326	0.0554	0.0160	0.0714	47 050	
100 000	2 300	0.34	3.90	0.908	305	0.0578	0.0140	0.0713	44 000	Minimum cost..
120 000	2 300	0.24	4.48	0.920	269	0.0610	0.0106	0.0716	38 700	

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the compression parts of the concrete are smaller; but, if medium steel is used, only very low working stresses should be allowed.

If medium steel, with an elastic limit of 35 000 lb. per sq. in., is used in a concrete having an ultimate strength of 2 300 lb. per sq. in., and a factor of safety of 4 is wanted, the working stress in the steel must be only, say, 9 000 lb. per sq. in., and in the concrete, say, 600 lb. per sq. in. As shown in Tables 3 and 5, it will be cheaper to use in the concrete a working stress of from 450 to 500 lb. per sq. in. with the 9 000 lb. per sq. in. in the steel.

In relation to T-beams, the speaker is in agreement with Captain Sewell. As steel having a low elastic limit does not utilize the full strength of the concrete in compression, even for a rectangular beam, the projecting flanges are not needed, and cannot act to decrease the dimensions of the beam. On the other hand, with steel having a high elastic limit, the full strength of the concrete is utilized in a beam of rectangular section; hence, the projecting flange will make it possible to increase the steel reinforcement without crushing the concrete. Therefore it will be proper to make $\frac{A}{b\ d} = \frac{1}{p}$, as proposed by the author, and then find h from Equation 7.

If the full width of the beam which can be considered is $B = b + 2\ c$, it is necessary to inquire whether or not $\frac{1}{p} \times \frac{b}{B}$ is greater than the percentages given in Table 5. With $p = 72$ and $\frac{B}{b} = 3$,

$$100 \times \frac{1}{72} \times \frac{1}{3} = 0.46\%,$$

hence the equation can be used for steel having an elastic limit up to 80 000 lb. per sq. in.

For steel having an elastic limit of 60 000 lb. per sq. in. the percentage in Table 5 is 0.80. Hence, for steel having an elastic limit up to 60 000 lb. per sq. in., the equation proposed by the author may be used if $\frac{B}{b} = \frac{1.39}{0.80} = 1\frac{3}{4}$, this is, if the projecting flange on each side of the beam is at least three-eighths of the width of the beam itself.*

In the foregoing the speaker has considered bending, but has not considered shear. The shear certainly has to be taken care of, as well as the bending; but, as it is evident that the cost of a well-designed web reinforcement is entirely independent of the depth of the beam, it is proper not to consider the web reinforcement when trying to find the most economical depth.

* If the neutral axis is below the projecting flange—which will generally be the case—this is a little on the unsafe side; hence it will be safer to allow the 60 000 lb. per sq. in. only if $\frac{B}{b}$ is, say, 2 or greater.

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The speaker considers the web reinforcement very important, and, without doubt, the ideal construction is with attached web members. As designed by the author, this seems to be effective and cheap.

It seems to be still an open question whether, with attached web members, it is justifiable to assume that total failure will occur before the steel is stressed appreciably beyond the elastic limit. The author explains very logically his reasons for this assumption, but the speaker doubts whether the increase in the strength of the steel, due to web reinforcement, is appreciable. It seems natural to assume that the concrete will be crushed very shortly after the stress in the steel has reached the yield point. In any event, the speaker considers it safer to assume the ultimate strength of the steel at the elastic limit until reliable experiments have shown that it is above that limit.

On page 261 the author says:

"It is certain that it can never be economical to use enough steel to cause the beam to fail first by crushing the concrete."

Table 5 shows that, for steel having a very high elastic limit, it is cheaper, and, even for steel having a lower elastic limit, it may be economical—in order to decrease the depth of the floor—to use so much steel that the beam fails first by crushing the concrete. Beam No. 13, in the lower part of Table 4, is the most economically designed of all the six beams in that series, both theoretically and practically, and it failed by the crushing of the concrete.

On page 267 the author's expression, "maximum allowable percentage of steel," is very misleading, as it leads one to believe that additional steel is dangerous, which is not the fact. Additional steel not only decreases the stresses in the steel, but, also, the stresses in the concrete, if the moment be assumed as constant.

Quite another matter, however, is the fact that it might be fatal in solving Equation 7 for d, using t_s as the elastic limit of the steel used, when a is larger than what the author calls the "maximum allowable percentage of steel." With excessive steel, the value of t_s must be taken as that where the percentage used is allowable. An example will illustrate this: Under the assumptions made, Table 5 shows that, for steel having an elastic limit of, say, 55 000 lb. per sq. in., the most economical percentage is 0.92; then $\frac{d}{\sqrt{m}} = 0.0480$, and $\frac{x}{\sqrt{m}} = 0.0798$.

If the steel is increased to 1.07%, the allowable value for t_s is 50 000 lb. per sq. in., $\frac{d}{\sqrt{m}} = 0.0469$, or 2.3% smaller than before: $\frac{x}{\sqrt{m}} = 0.0830$, or 4.0% higher than before.

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If the steel is decreased to 0.70%, Equation 7, $m = h\,a\,d\,t_s$, gives:

$$m = 0.875 \times 0.70 \times d^2 \times \frac{55\,000}{100} = 337\,d^2;$$

$$\frac{d}{\sqrt{m}} = 0.0546, \text{ or } 13.8\% \text{ higher than in the first case;}$$

$$\frac{x}{\sqrt{m}} = 0.0546\,(1 + 0.72 \times 0.70) = 0.0821, \text{ or } 2.9\% \text{ higher than}$$

in the first case.

It will be seen, therefore, that the most economical results are secured by using the percentages given in Table 5. Any smaller percentages will increase the cost as well as the depth of the beam. Any larger percentages will increase the cost, but decrease the depth of the beam, and the latter, under certain circumstances, may be of great value.

For these reasons, therefore, the author is incorrect in stating (page 267 that "theoretical economy, based on relative costs, is not attainable." Theoretical economy, based on relative costs, is reached when what the author calls the maximum allowable percentages are used. The author is also incorrect in stating that any more than the economical percentage is wasted. As just shown, an increase in the steel increases the strength of the beam; consequently, it is not wasted.

With the assumption, made in Table 5, that the ultimate strength of the concrete in compression is 2 300 lb. per sq. in., the most economical percentage of steel for t_s smaller than 42 300 lb. per sq. in. is 1.4. $\frac{d}{\sqrt{m}}$ will be seen to vary approximately in proportion to t_s, according to two straight lines which intersect at a minimum point where $t_s = 42\,300$.

The following equation, proposed by the speaker, is for a rectangular beam or floor slab:

$$d = \sqrt{m}\,(\alpha + \beta\,t_s),$$

in which α and β have different values for various values of F. Assuming $F = 2\,300$, for values of t_s greater than 42 300, $\alpha = +0.0360$, and $\beta = +0.218 \times 10^{-6}$. For values of t_s smaller than 42 300, $\alpha = +0.074$, and $\beta = -0.69 \times 10^{-6}$. For other values of F, α and β will have other values, and the point of intersection of the two curves will change. To find the point of intersection for other values of F, the equation,

$$\alpha\,t_s = 0.57\,F\,\frac{d}{\beta},$$

should be used, as the point of intersection is where

$$d = p\,a, \text{ or } \frac{100\,a}{d} = \frac{100}{p} = 1.39.$$

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$$\frac{t_s}{F} = \frac{57}{1.39\ \beta} = \frac{41}{\beta};$$

but

$$\beta = 1 + \tfrac{1}{15} \times \frac{t_s}{F}.$$

hence

$$\frac{t_s}{F} = 18.4.$$

The speaker's formula, therefore, will be:

1.—For t_s smaller than 18.4 F: Use 1.4% of steel and fix the depth of the beam in accordance with Equation 7, that is, $m = h\ a\ d\ t_s$, substituting for h, 0.85, and for a, $\frac{1.4}{100}\ d$; hence,

$$d = \sqrt{\frac{84\ m}{t_s}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ A$$

2.—For t_s larger than 18.4 F: Make depth of beam

$$d = \sqrt{m}\,(\alpha + \beta \times 10^{-6} t_s) \ldots\ldots\ldots\ldots\ldots\ B$$

The values of α and β, for various values of F are:

F.	α.	β.
1 900	0.040	0.282
2 100	0.038	0.244
2 300	0.036	0.218
2 500	0.034	0.200

The percentage of steel can then be fixed from Equation 7, using $h = 0.85$ and $m = 0.85\ a\ d\ t_s$, or

$$\frac{100\ a}{d} = \frac{100\ m}{0.85\ d^2\ t_s} = \frac{118\ m}{d^2 t_s}$$

$$= \frac{118}{t_s\,(\alpha + \beta \times 10^{-6} t_s)^2}$$

In these formulas:

m = the maximum moment per inch multiplied by the factor of safety required;

a = the area of the reinforcing rods per inch of beam;

d = the depth of the beam from the center line of the steel reinforcement to the ultimate fiber;

F = the ultimate compressive strength of the concrete;

t_s = the ultimate strength of the steel reinforcement (and for this the speaker proposes to use the elastic limit of the

* This, it will be observed, is simply another form of Thacher's formula.

steel until it is plainly shown that a higher value is permissible). Mr. Forchhammer.

Example 1.—A beam, 8 in. wide, is subject to a maximum bending moment of 100 000 in-lb. A factor of safety of 4 is required. The ultimate strength of the concrete is 2 200 lb. per sq. in., and the elastic limit of the steel is 30 000 lb. per sq. in.

$$m = 4 \times 100\,000 \div 8 = 50\,000.$$

As $\frac{t_c}{F} = \frac{30\,000}{2\,200} = 13.6$, and is less than 18.4, Equation A must be used.

Percentage of steel = 1.4.

$$d = \sqrt{\frac{84 \times 50\,000}{30\,000}} = 11.82 \text{ in., say } 12 \text{ in.}$$

Example 2.—A beam, 6 in. wide, is subject to a maximum bending moment of 72 000 in-lb. A factor of safety of 5 is required. The ultimate strength of the concrete is 2 300 lb. per sq. in., and the elastic limit of the steel is 60 000 lb. per sq. in.

$$m = 5 \times 72\,000 \div 6 = 60\,000.$$

As $\frac{t_s}{F} = \frac{60\,000}{2\,300} = 26.1$, and is larger than 18.4, Equation B must be used.

$$d = \sqrt{60\,000}\,(0.036 + 0.218 \times 10^{-6} \times 60\,000) = 12.0 \text{ in.}$$

$$\frac{100\,a}{d} = \frac{118 \times 60\,000}{144 \times 60\,000} = 0.82 \text{ per cent.}$$

If it is of importance to decrease the depth of the beam as much as possible, $\frac{t_s}{F}$ should be assumed as equal to 18.4. Then $t_s = 18.4 \times 2\,300 = 42\,300$. d can then be calculated either from Equation A or Equation B.

Using Equation A:

$$d = \sqrt{\frac{84 \times 60\,000}{42\,300}} = 10.9 \text{ in.}; \text{ and } \frac{100\,a}{d} = 1.4 \text{ per cent.}$$

Using Equation B:

$$d = \sqrt{60\,000}\,(0.036 + 0.218 \times 10^{-6} \times 42\,300) = 11 \text{ in., and}$$

$$\frac{100\,a}{d} = \frac{118 \times 60\,000}{121 \times 42\,300} = 1.39 \text{ per cent.}$$

The results from these examples will be seen to check very closely with the results given in Table 5, and the speaker concludes that the equations proposed are sufficiently close approximations.

The speaker does not claim that the constants used for p, e, F, t_s, etc., will prove to be just the right ones. They may vary much, under different conditions, and only further experiments can fix their values.

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To avoid any misunderstanding, the speaker wishes to state, finally, that the formulas proposed herein can only be applied where beams or floor slabs are exposed to actual bending. Wherever the arch action in the concrete is appreciable other formulas may be applied.

Mr. French.

ARTHUR W. FRENCH, M. AM. SOC. C. E. (by letter).—This paper is full of interest for designers of reinforced concrete, and the author will receive the gratitude of all for his excellent work.

Considering the many theories and formulas for the design of reinforced concrete beams, the greatest difference of opinion seems to be in the matter of using safe stresses or ultimate stresses in the steel and concrete.

The author proposes that the design be based upon the ultimate strength of the beam. The writer prefers the use of safe working stresses in the materials.

It is common practice to design wooden and metal beams on the safe working stresses. Our formulas for flexure are theoretically correct for such beams when the stresses are within the elastic limits. Outside of these limits, although the same formulas are used, it is recognized that they are purely empirical, and that they depend upon tests for the values of the constants.

It is not impossible to construct formulas, for the ultimate strength of beams of homogeneous materials, based upon the properties of the stress-strain diagrams of the materials, but it is generally considered impractical. If the stress-strain diagram of concrete were not quite such a smooth and attractive curve, often with a marked resemblance to a parabola, it is doubtful whether anyone would attempt a theoretical formula for the ultimate strength of a beam of this material reinforced with another material.

An examination of many diagrams for concrete will show considerable variation in the relation between strain and stress as the stress increases from zero to the breaking point. The lines often resemble the parabola with its axis at the breaking point, but, with the richer concretes, the lines become more nearly straight. In all these diagrams, if the attention be confined to the part of the line from zero up to any stress that would be used as a working stress, it will be seen to be sensibly straight. Moreover, the elastic deformation for stresses from 0 to 500 or 1 000 lb. has been well determined for many concretes, and probably with greater accuracy than for stresses near the breaking stresses.

With the assumption of a constant ratio of stress to strain within the limits of the working stresses (and the assumption is close to the truth and slightly on the safe side), the design may be made with as much certainty as in the case of wooden or steel beams. Let the ultimate strength of the beams be a matter of test and the results be expressed by an empirical formula.

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Fig. 28 gives the formulas used for safe working stresses and a graphical solution showing the relations between the unit stresses in the steel and concrete for various percentages of reinforcement.

It is important to note that the exact position of the neutral axis is of little importance as far as the strength, calculated from the steel element, is concerned. The effective arm of the couple changes very little with widely differing assumptions as to the value of E_c, or of the shape of the diagram for deformation of the concrete, and but little with the percentage of reinforcement.

The depth of concrete under compression and the total compression in the concrete, however, is affected greatly by the position of the neutral axis and also by the assumed shape of the diagram for concrete. Fig. 29 gives the formulas for the assumption of a full parabola for the shape of the stress-strain diagram between the limits, zero and the working stress, and considers the coefficient of elasticity to be 3 000 000 and 2 000 000 at the origin of the curve.

Fig. 30 is also for the parabolic distribution of stress, but is based on coefficients of elasticity of 3 000 000 and 2 000 000 at the working stress. The assumption in the last case is equivalent to an initial coefficient of either 6 000 000 or 4 000 000.

The absurdity of assuming a full parabola for the stress distribution within a working limit of from 500 to 750 lb. has often been pointed out. Formulas based upon such an assumption may be made to give any desired results if the E is chosen properly, but they must be considered as purely empirical, and the writer would prefer an empirical formula of the correct form.

The writer cannot agree with the proposed formula of Mr. Wason for general application. The assumption of the neutral axis in the center of the beam is in close agreement with theory for the percentage of steel in the examples, and, used by its author or other expert, within certain limits, it gives good results. It is certainly the simplest formula proposed, and for about 1% of steel reinforcement, is of great value. Its use is equivalent to allowing 750 lb. stress upon the outer fibers of the concrete if the straight-line formula is used as a test. Probably many beams which are calculated by certain formulas to give 500 lb. stress have nearer 750 lb. if tested by the straight-line formula.

Practical experience with many structures of concrete tested up to 2 000 lb. per sq. in. in straight compression, as well as laboratory tests of such beams, seems to show that if designed by the straight-line formula, with from 600 to 750 lb. stress in the concrete, they will possess ample factors of safety. It may sound better to talk of 500 lb., rather than a higher stress, but the truth is the thing desired.

The use of the straight-line formula for working stresses will give lower percentages of steel allowed in the section, or if high

Mr. French.

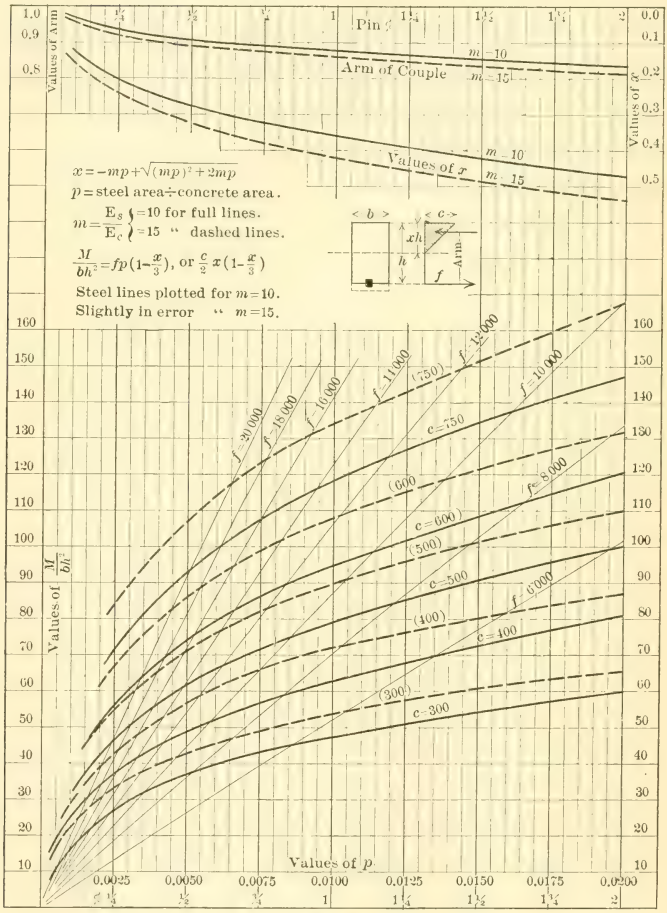

FIG. 28.

Mr. French

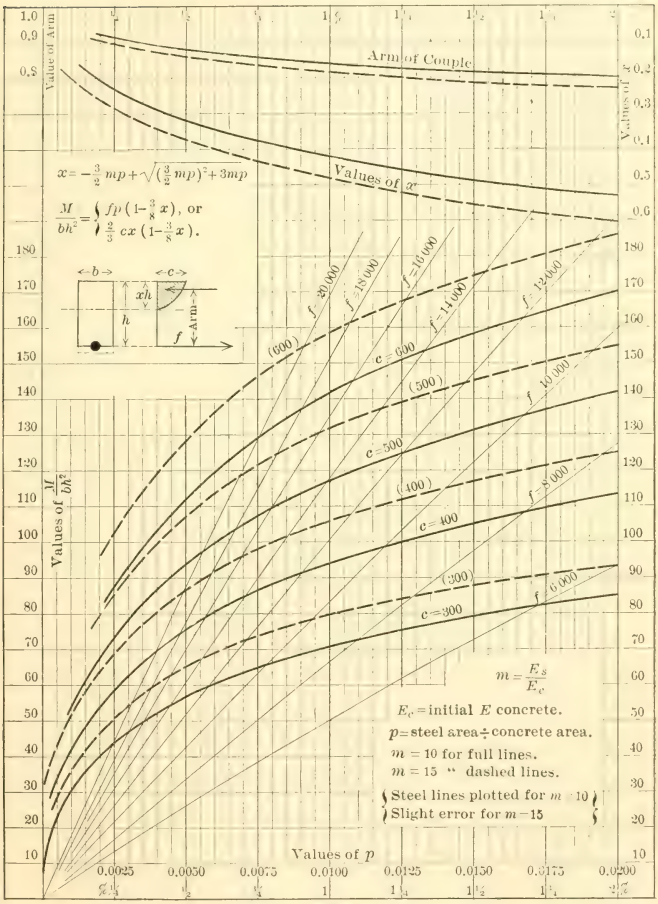

FIG. 29.

Mr French.

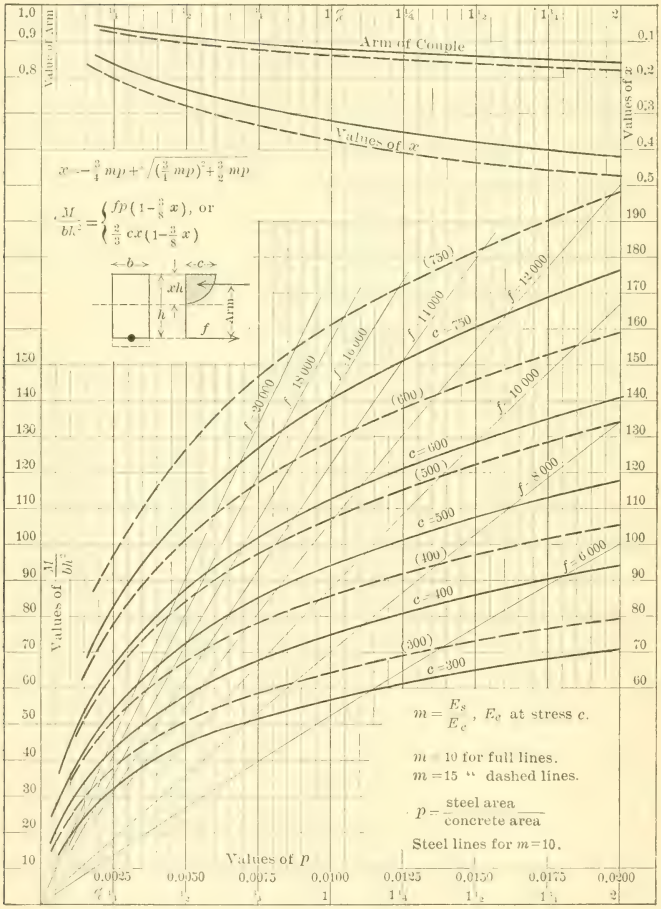

FIG. 30.

percentages are used, the allowable stress in the steel is reduced automatically. As an example of the use of Fig. 28, if the stress in the concrete is to be limited to 600 lb., and $\frac{E_s}{E_c}$ is taken as 15, and the working stress permissible in the steel is 16 000 lb., the value of $\frac{M}{b\,h^2}$ will be 72, for $p = 0.005$. If p is 0.0065, the stresses will be 600 and 16 000 lb., and the value of $M \div b\,h^2$ will be 94. Where conditions demand, it may be wise to use higher percentages of steel, although this is not theoretically economical. Thus, if $p = 0.02$, the allowable stress in the concrete, of 600 lb., will determine the value of $M \div b\,h^2$ as 132, and the corresponding stress in the steel will be about 8 000 lb. Mr. French.

The natural limitations in the use of reinforced concrete beams and the desire for long spans or heavy load-carrying beams are constant temptations to over-steel the beams, and some formula which controls automatically the relation between the areas of steel and the stresses therein seems to be very desirable.

Shear reinforcement is shown to be needed in most beams, both by analysis and by tests, and it is well that increasing attention is being given to the subject. In view of the comparatively small amount of steel needed to insure safety from failure by shear, it is the writer's practice to be rather liberal with such reinforcement. If stirrups are to be used at all, the added expense of furnishing and placing a liberal supply is small.

Undoubtedly the most effective form of shear reinforcement is that of rods attached rigidly to the tension bars, sloping up toward the supports, and extending well to the top, or even extending into the slab. The one form of patented reinforcement which well fills the requirement of rigid attachment does not always easily supply the requisite number and length of shear rods where they are needed most. Other forms of rigidly attached rods seem to be clumsy and expensive. If the shear rods are to be loose stirrups, they should be set vertically. Ease in placing the concrete and in providing the desired number of rods of the proper length cause the writer to prefer, in practice, the loose vertical shear rods.

There would seem to be no reason why the width, b', of a slab assumed to act as a compression flange, should be limited to three times the width of the stem, b, if the proper reinforcement for shear at the section between the wings and stem be provided. Without reinforcement, the limit of $b' = 3\,b$ is well taken.

Irving P. Church, Assoc. Am. Soc. C. E. (by letter).—In making a contribution to the discussion on Captain Sewell's valuable paper, the writer has in mind a brief, though fairly systematic, investigation of the relation between the cost and the dimensions, stresses, Mr. Church.

Mr. Church. etc., of the ordinary concrete-steel beam or slab of rectangular section, with reinforcement on the tension side only. This treatment will be based on the assumption of the straight-line, stress-strain diagram for the concrete, and the tension of the concrete will be neglected (all loads and reactions being perpendicular to the beam and in the same plane). The notation adopted will be the same as that already used in the paper, but, for convenience, two additional symbols will be used, viz., r and e, representing the ratios, $\frac{T}{F}$ and $\frac{E_s}{E_c}$, respectively. The latter ratio will be taken as constant.

Since the distances, y_1 and y_2, are superfluous, as far as formulas for actual design are concerned, they will be eliminated at an early stage, while the compressive stress, F, in the "outer fiber" of the concrete need not be directly expressed in the final formulas, being replaceable when desired by the equivalent quotient, $\frac{T}{r}$. Again, T and F may have any values within reasonable limits, and the ratio, $\frac{T}{F}$, is not taken as equal to $\frac{f_s}{0.8 f_c}$, necessarily.

The author's Equations 1 and 2 still hold in the present connection, and from them is derived:

$$y_1 = \frac{e\,d}{r+e} \qquad \text{(1c)}$$

The centroid of the compressive stresses being at a distance of $\frac{2}{3} y_1$ from the neutral axis, the arm of the couple formed by the total stress, $a\,b\,T$, in the steel, and, $\frac{F\,b\,y_1}{2}$, in the concrete, is $d - \frac{y_1}{3}$, and hence

$$a\,b\,T\left(d - \frac{y_1}{3}\right) = M \qquad \text{(2c)}$$

The equality of the total stresses mentioned gives rise to the relation, $\frac{b\,F}{2}\,y_1 = a\,b\,T$, that is, $a\,b = \frac{b\,y_1}{2\,r}$, which, combined with Equation 1c, leads to

$$2\,r\,a\,(r+e) = d\,e \qquad \text{(3c)}$$

If y_1 is now eliminated from Equation 2c, by the aid of Equation 1c, and if for all subsequent work a value of unity be assumed for the width of the beam, so that the bending moment, M, becomes $m = \frac{w\,l^2}{8}$ (w being the uniform load per square inch of upper surface of the beam and l the span), there is obtained

$$3\,m\,(r+e) = (3\,r + 2\,e)\,T\,a\,d \qquad \text{(4c)}$$

Mr. Church.

Equations 3*c* and 4*c* are the only relations needed for design (aside from considerations of shearing stresses) for a beam of rectangular section with reinforcement on the tension side only. They constitute two independent equations, from which, if all the quantities concerned except two are given or assumed, these two may be determined, and hence should be regarded as constants.

If, however, all the quantities concerned in these two equations be given or assumed except three, these three are not determinate, but are variables; and by the aid of Equations 3*c* and 4*c*, a relation may be obtained between any two of the three variables; that is, any one of them may be expressed as a function of either of the other two.

For use in subsequent work the following relations, all derivable from Equations 3*c* and 4*c*, are here appended:

$$a = \frac{d\,e}{2\,r\,(r+e)} \qquad (5c)$$

$$a = \sqrt{\frac{3\,m\,e}{2\,r\,T\,(3\,r+2\,e)}} \qquad (6c)$$

$$d = (r+e)\sqrt{\frac{6\,m\,r}{(3\,r+2\,e)\,T\,e}} \qquad (7c)$$

The cost of a beam (of unit width), of fixed span and maximum bending moment, m, will now be expressed in terms of various variables, and the conditions of minimum cost investigated.

From the nature of the case, the only quantities which are not fixed at the outset, and hence would be available as variables, are four in number, viz., a, d, r, and T; so that four groups, of three in a group, may be selected, in turn, each group comprising three variables (m is treated as a constant).

As to the question of cost, consider only the items, (3) and (4) (mentioned on page 264 of the paper), assuming the cost of a cubic foot of steel to be equal to p times that of a cubic foot of concrete, and denoting by x a number proportional to the total cost of the two materials. Then, as in the paper,

$$x = p\,a + d \qquad (8c)$$

As a first group of variables take a, d, and T. Then, from Equations 5*c* and 8*c*, is found:

$$x = \left[\frac{p\,e}{2\,r\,(r+e)} + 1\right] d \qquad (9c)$$

which gives x as a function of the one variable, d.

Here it is seen that x is directly proportional to d, so that there is no mathematical minimum. Hence, for practical design, we have to take simply as small a value for d as may be consistent with a maximum safe value for T (one of the other two variables), while also ensuring a safe value for F, as implied in the assumed value of r.

Mr. Church. For example, let $e = 16$, $r = 20$, $w = 8$ lb. per sq. in., $l = 100$ in. (so that $m = 10\,000$ in-lb.), and $T = 16\,000$ lb. per sq. in. With these values, then, from Equation 6*c*,

$$a = \sqrt{\frac{30\,000 \times 16}{40 \times 16\,000\,[60 + 32]}} = \sqrt{0.00815} = 0.0903 \text{ sq. in.},$$

and for d, from Equation 5*c*,

$$d = \frac{36 \times 2 \times 20 \times 0.0903}{16} = 8.12 \text{ in.}$$

This would make the percentage of steel $\frac{0.0903}{8.12} \cdot \frac{1}{100} = 1.10\%$ (referred to the concrete above the steel); and the value of F, implied in the assumption of $r = 20$, is $16\,000 \div 20 = 800$ lb. per sq. in.

Secondly, take r, a, and T, as variables. Equation 9*c* will serve in this case, also; that is,

$$x = \left[\frac{p\,e}{2\,r\,(r+e)} + 1\right] d \qquad (9c)$$

giving x as a function of the one variable, r, d being a constant in this case. Evidently, x decreases with an increasing r. Here, again, there is no mathematical minimum for x (for any positive value of r) so that a practical use of the relation would be to take as large a value for r as would be consistent with a maximum safe value for T. For instance, with $T = 16\,000$ lb. per sq. in., $d = 10$ in., $e = 16$, and $m = 10\,000$ in-lb., it is found, from Equation 7*c* (solving by trial, since this proves to be a cubic equation), that the value of $r = 26.3$; which, in Equation 5*c*, gives a value of 0.0719 sq. in. of steel for a, implying 0.719% of steel (if the steel is compared with the concrete situated above it). With $r = 26.3$, the value of F would be 609 lb. per sq. in. With values of r smaller than 26.3, the cost would be greater, and both T and F would have smaller values than the above (in this case of constant d, etc., and variable r, a, and T); and neither material would be worked to its full (safe) strength.

Thirdly (and this is the most important and practical case), consider as the group of three variables, d, a, and r. If, in Equation 8*c*, there are substituted values for a and d from Equations 6*c* and 7*c*, there is obtained an expression for x as a function of the one variable, r, viz.,

$$x = \sqrt{\frac{3\,m}{2\,T\,e}}\left[\frac{p\,e + 2\,(r+e)\,r}{\sqrt{r\,(3\,r+2\,e)}}\right] \qquad (10c)$$

To obtain a value of r for minimum cost, the differential coefficient of x with respect to r should be placed equal to zero, and the resulting equation solved for r; but this would be found to lead to an equation of such high degree that a simpler plan is to compute the

value of x for each of a number of values of r and note the occur- Mr. Church.
rence of a minimum x.

The writer has done this for seven values of r, ranging from 4 to 50, in an example in which there is given $m = 10\,000$ in-lb., $T = 16\,000$ lb. per sq. in., and $e = 16$; while p may be taken as 75 (that is, let 20 cents per cu. ft. be the cost of the concrete, and \$15 that of the steel).

Since the variable part of the expression for x is the factor in the bracket, let the other (constant) factor be denoted by C. The following values for x have been computed:

For $r = 4$, $x = 102.6\ C$
" $r = 9$, $x = 71.6\ C$
" $r = 16$, $x = 62.2\ C$
" $r = 20$, $x = 61.6\ C$
" $r = 25$, $x = 62.9\ C$
" $r = 36$, $x = 69.6\ C$
" $r = 50$, $x = 81.8\ C$

It is seen that in this case x has a mathematical minimum, and that it is reached for a value of about 18 for r; and it is also noticeable that as r changes from 15 to 25 the variation in the cost is comparatively small.

Therefore, it may be said that in this example the cost is a minimum for $r = 18$.* Hence, with $T = 16\,000$ lb. per sq. in. the value of F would be 888 lb. per sq. in. If it were desired that the stress in the concrete should not exceed, say, 600 lb. per sq. in., this could be secured by adopting for r the value, 26.6, and the resulting cost would be but little in excess of that (the minimum) corresponding to $r = 18$.

As to the values of a and d, corresponding to $r = 18$ in this example, it is found, from Equation 7*c*, that $d = 7.54$ in.; and then, from Equation 3*c*, $a = 0.098$ sq. in. of steel; which is 1.31% of the part of the concrete above the steel.

As to the only remaining group of three variables, viz., d, T and r, it may be noted that the term, $p\,a$, in the expression for cost, $x = p\,a + d$, is constant (since a is assumed in this case), and that, consequently, the variable part of x is directly proportional to the height, d, and will be a practical minimum when d is made as small as possible consistent with a maximum safe value for T, one of the other variables. Hence, this case is virtually the same as one already treated. When the d has been computed for a proper T the corresponding values of r, and later that of F, are easily found.

Although the foregoing has been based on the simple assumptions of the straight-line, stress-strain relation, it is thought that

* A later solution by the calculus gives 19.3 instead of 18.

Mr. Church. the results would not have been very different if the parabolic form of curve had been used.

Mr. Leffler. B. R. Leffler, Assoc. M. Am. Soc. C. E. (by letter).—The writer has had in mind for some time the publication of a paper on steel-concrete formulas, but Captain Sewell's paper has served a somewhat similar purpose. The writer wishes to add the following remarks in the way of a partial discussion, and addition; they apply to the subject in general.

Nearly all writers have adopted the ultimate-strength method of designing, but the writer believes this is a mistake. The evolution of rational designing shows a continual substitution of simple formulas of wide application for complicated ones of narrow application. For instance, there was a time when column formulas were entirely empirical. At present they are rational in form, with, perhaps, empirical constants. In fact, nearly the whole science of the mechanics of materials is derived from Hooke's Law—a simple law of wide application.

The common beam formula has been hotly disputed in the past, because it did not represent the actual conditions at the ultimate strength period. It would seem that reinforced concrete formulas are passing through a similar period of development; and that, finally, a simple formula, having a straight line in its stress-strain diagram, will supersede them all.

The writer's experience in reinforced concrete has been confined to the simple beam, and arches.

It is rather astonishing that, in the voluminous discussions of the past few years, no one has attempted to present a rational method of designing a reinforced concrete section for combined thrust and moment, on the supposition that steel takes tension only and concrete compression only. Edwin Thacher, M. Am. Soc. C. E., presented a formula, some years ago, in which concrete is supposed to take tension.*

Writers who adopt the ultimate-strength method of designing seem to be unaware of the impassable difficulties they create in a design for combined thrust and moment. They thus forever bar the rational design of reinforced concrete arches. As is well known, at any section of an arch there may be combined thrust and moment.

It may be well to state that all formulas and methods of designing arches are based on the assumption of a straight line in the stress-strain diagram.

The writer has before him a pamphlet—widely circulated throughout the Middle West—which purports to apply the ultimate-strength method to arches. The author of the pamphlet uses the common elastic theory for locating the pressure curve—a parabola

* *Engineering News*, September 21st, 1899.

in the case cited—but, in designing the individual sections for combined thrust and moment, he attempts to use the ultimate-strength method. These are some of the vagaries in the development of reinforced concrete design. Mr. Leffler.

In designing the sections of an arch, the writer first used Thacher's formulas,* but abandoned these in favor of a method in which concrete takes no tension. He presents the following as an easy and rational method of designing a section for combined thrust and moment.

Let Fig. 31 represent two short blocks, having steel on one side, as shown, and subject to an eccentric compressive load. This is a typical case in arch design. Then, granting a straight line in the stress-strain diagram, no tension will occur in the steel as long as the load acts within the middle third of the concrete section. The small strip of concrete on one side of the steel is neglected.

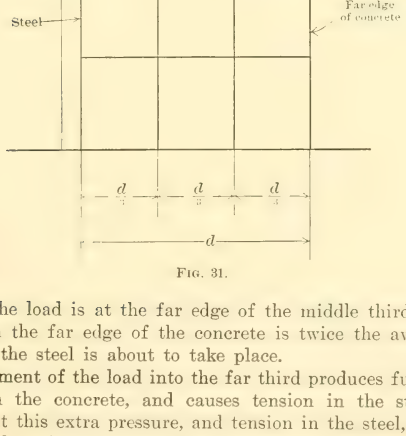

FIG. 31.

When the load is at the far edge of the middle third, the compression on the far edge of the concrete is twice the average, and tension in the steel is about to take place.

A movement of the load into the far third produces further compression in the concrete, and causes tension in the steel. It is evident that this extra pressure, and tension in the steel, is directly proportional to the distance of the point of application of the load beyond the far edge of the middle third.

For simple beams, the writer uses the straight-line formulas, as given in "Concrete, Plain and Reinforced," by Taylor and Thompson.

* *Transactions*, Am. Soc. C. E., Vol. LV, p. 188.

Mr. Leffler. The following is the writer's simple method of designing for combined thrust and moment:

The moment, for extra pressure at the far edge of the concrete and for tension in the steel, is equal to the distance of the point of application of the load beyond the far edge of the middle third, multiplied by the load. Use this moment as if it occurred in a simple beam, determining the extra pressure on the concrete, and tension in the steel. Add the extra pressure to twice the average pressure for the total pressure on the concrete at the far edge.

It is well to remember that the middle-third theory is based on a straight line in the stress-strain diagram, and cannot be used in the ultimate-strength method of designing.

The writer would like to see an advocate of the ultimate-strength method of designing attempt to show when steel takes tension in the above case!

The assumption of a straight line in the stress-strain diagram leads to simple formulas of wide application, and hence is bound to prevail. Of course, in this assumption, working stresses should be used, and not ultimate stresses.

It seems to the writer that the present field of engineering literature on reinforced concrete is being turned into a vast mathematical gymnasium. An engineer is not primarily a mathematician, but only incidentally so.

Mr. Hill. GEORGE HILL, M. AM. SOC. C. E. (by letter).—Although reinforced concrete has been in use now for some few years, and the literature on the subject is becoming voluminous, a lack of agreement is still found in regard to facts. Relatively few tests have been made, and there are imperfections in the observations, tending to minimize the value of these tests, the majority of which have been for commercial rather than engineering purposes.

In glancing over the literature on the subject, the reader will notice that as a writer gains experience in the actual use of reinforced concrete his views regarding it change. He is more wary in using indiscriminate tests, is apt to make more for himself, and is less certain that he can solve all problems theoretically. This would indicate the desirability of less work with pencil and paper and far more work with a testing machine, before the application of the differential calculus to the results.

In certain tests made by the writer* he was struck by the similarity, in reasonably good slabs, of what might be called the load curve. All these tests were made with a portable hydraulic testing machine having a recording apparatus. Pains were taken to secure a reasonably uniform application of the load, and to record the deflection constantly, so that for each test there was a graphical

* *Transactions*, Am. Soc. C. E., Vol. XXXIX, p. 617.

representation which spoke directly to the eye and conveyed lessons which would otherwise have been lost unless considerable labor were expended in plotting the results. All the load curves indicated an elastic limit point for the combination, which also appeared in the deflection curve. In the writer's opinion, tests which stop short of the destruction of the object tested are of no value, and these constitute the bulk of the public tests during the past five years. Tests which do not indicate clearly the behavior of the piece under each increment of load, and those made for some especial purpose, without photographs, are of but little value, as the observations are open to dispute; therefore one can understand the author's desire for further tests. Mr. Hill.

Another serious difficulty is the lack of uniformity in the concrete. In the writer's opinion, therefore, the two most fruitful subjects for present investigation are the production of a concrete by commercial methods having fairly uniform characteristics which can be predetermined, and the testing to destruction of reinforced concrete members, under conditions which will secure graphic records of the load, its rate of application, the deformation at each instant, and the appearance of the piece tested at certain critical periods.

The author, on page 263, states as follows:

"Therefore it seems justifiable to assume that a formula such as Equation 6 can be made just as accurate as any of the forms of Equations 1 to 5, especially if its constants be determined from tests designed with that end in view."

With this, the writer is in full accord. To a considerable extent, it supports the view he expressed in his paper* presented in April, 1898. While engineers may feel that it is too simple a solution for men of their mathematical attainments, yet, as they are charged with the duty of getting full value for their employers' time as well as their own, they should realize that the present state of the art admits of nothing better.

The writer doubts very much the value of the theoretical discussion of maximum economy by the methods of the calculus. Not only does the cost of the aggregate vary in different localities, but it varies constantly in the same locality. Owing to the beneficent rule of the labor unions, one may in one locality pay for labor at $1.75 per day, to-day, and six months later be required to pay $4 per day for the same work. During one month it may be found that one form of deformed steel bar can be had cheapest, and the next month some other form. It may be permitted on one job to use a certain style of form, or centering, which is relatively inexpensive and can be used repeatedly, but the next job may be so different that

* *Transactions*, Am. Soc. C. E., Vol. XXXIX, p. 617.

Mr. Hill. none of the former centering is available. Therefore, it follows that whatever may be the most advantageous for one job may be very uneconomical for the following one. The writer believes that no formula can be devised which will cover these conditions, and be used by a busy man.

In the ninety-four tests published in the writer's paper, previously referred to, he observed in no case any necessity for web reinforcement, and, in properly designed commercial structures, he doubts the necessity or desirability of such reinforcement. In relatively deep and slender T-beams it may be necessary, but it is questionable whether or not it is commercially desirable to design such beams.

The writer is heartily in accord with the following statements by the author: on page 253, in regard to the impression of the extreme accuracy of the complicated formula, which is not at all justified; on page 257, in regard to the desirability of taking conservative values for the maximum allowable stresses; on page 261, in regard to the experimental determination of the maximum allowable percentage of reinforcement; on page 263, heretofore referred to; on page 267, that the best that can be done is to use as large a percentage of steel as the concrete will stand; on page 278, that a mechanical bond is to be preferred to simple adhesion; on page 284, that it is of importance that the floor slab and the part of the beam or girder below it should be bonded together; and paragraph 6 on page 286.

In regard to the statement, on page 281, concerning the dehydration of cement, the facts may be as stated by the author; the conclusion that this damaged material must be removed does not follow, for, if the tensile strength of the cement is neglected in the computations, this material is only used as a protection for the steel, and if it performs that function it might as well be left. It will perform that function if a wire netting or other similar substance be used for the sole purpose of retaining the cement around the reinforcement. It is to be earnestly hoped that this paper will attain its main object, as explained in the last paragraph on page 286.

Engineers who design reinforced concrete should come to an agreement in regard to the most desirable method of making tests; in regard to practicable formulas which can be applied in design; in regard to formulas applicable to floor slabs with numerous supports; in regard to the shrinkage of slabs and walls in setting; in regard to a provision for either the elimination or localization of cracks; and in regard to the commercial limitations which, in general, should apply in the design and construction of reinforced concrete buildings.

It would be hard to imagine any better demonstration of the

need of an agreement in regard to making tests than that given in the discussion of this paper by Mr. Goodrich, and the results of tests published in the engineering press. In each case there seems to be a failure to describe certain conditions in the loading, or the omission of other elements, which would explain the great diversity in results, and harmonize what appear to be diametrically opposed conclusions. These tests give certain general data, and then draw certain conclusions; but, one series of tests will be made by applying a concentrated load at one point in the beam; another series will be made with a different style of reinforcement, with a load applied at two points in the beam; a third series will be made with the load normally uniformly distributed, and using virtual knife-edges for the support of the beam; and still another will be made with a normally uniformly distributed load carried on broad supports, and extended over the supports in such a way as to produce a partially fixed condition of the beam end. Mr. Hill.

In some cases, pig iron is used; in other cases, bags of cement; in still other cases, bricks; while, in a few cases, hydraulic pressure is applied. In nearly all cases the observations of deflection are defective. In none of the published cases are there shown any charts giving the rate of application of the load, its amount, and the corresponding deflections. The tests almost always neglect to give the weight per cubic foot of the material tested, and of its component parts, and they very generally neglect to give specific information in regard to the aggregate and the percentage of the various sized particles composing it.

It follows, therefore, that, in spite of the mass of information gradually being accumulated, much of it is so deficient in detail as to be unavailable. There exists an urgent need for a series of tests, conducted by some independent party, using two or three different kinds of reinforcement, using varying amounts of reinforcement variously disposed, and with different styles of loading and different characters and proportions of aggregate.

Were a series of tests made, it is probable that the reference to arch action, as a means of explaining the large carrying capacity of continuous beams and girders, would be omitted. In fact, it is hard to understand why reference should be made to arch action when, confessedly, engineers use as much steel as is necessary to develop the full compressive resistance of the concrete; and the increased carrying capacity of the beams can almost always be explained, either on the assumption that the factor of safety is reduced to 1, or by applying the accepted formulas for beams continuous over several supports. Mr. Goodrich mentioned many tests which had been made by him. These tests might be of very great value to the profession at large were they fully described, and it is to be hoped that he will give a complete record of them.

Mr. Hill. When the number of tests on the metallic materials of construction is compared with the tests on which practice in regard to reinforced concrete is based, engineers will realize on what meager data they are building, and will probably agree with the writer that, in spite of the relatively large volume of theoretical discussion, they are really dependent on empirical results almost entirely.

Mr. Shearwood. F. P. SHEARWOOD, M. AM. SOC. C. E. (by letter).—Captain Sewell's advocacy of a system which eliminates as far as possible all uncertainties in this popular mode of construction will recommend itself to all.

Although the various formulas for computing the strength of beams do not differ greatly in their final results, the many discussions on the subject, by their very existence, prove that the distribution of the stresses (especially of the so-called shearing stresses) is still an unsolved problem, a clear conception of which is not yet reached.

Tests on specially prepared specimens, or on such as have been carefully watched during construction, do not by any means fully furnish the desired information, for until the stresses in a reinforced beam can be analyzed with at least the same degree of confidence in the result as is permissible in steel construction, this method must give occasion for skepticism.

The writer believes that tests, almost invariably, have been made for a single loading, and it is as yet probably unknown whether beams of this construction do not deteriorate under frequent loadings. Many authorities seem to admit that the concrete in the reinforced or tension side of a beam is strained beyond its strength when the beam is loaded to its working capacity, therefore it follows naturally that it is more or less injured, and possibly serious harm will occur with frequent loadings.

From the very small deflections recorded under test loads on these beams it appears to be evident that the steel reinforcement is not performing the work for which it was designed, and therefore the concrete is performing a duty for which it is not safely capable, and, by being stressed repeatedly to its ultimate strength, it is likely to be fractured. How the destruction of the tension value in the concrete will affect its adhesive qualities should be ascertained before reliance is placed on the permanency of reinforced concrete.

A series of experiments to discover whether reinforced concrete can fail by fatigue, or to find out at what unit stress concrete can be strained when reinforced, without deterioration, would be a valuable addition to the useful knowledge on this subject.

Mr. Merriman. MANSFIELD MERRIMAN, M. AM. SOC. C. E. (by letter).—The author's method does not seem to be a satisfactory one for the design of beams, because his curve giving the distribution of stresses

above the neutral axis was deduced from tests in which concrete was ruptured under compression. While his Equations 1 to 5 may apply very well to cases of the rupture of beams, they seem to be inapplicable to problems of design, since the stress-strain curve for rupture is very different from that which prevails when the concrete is stressed only to such values as are allowed by specifications. The unit stresses that appear in these formulas are those of the ultimate compressive strength of the concrete and the elastic limit of the steel. In designing a beam, however, the allowable compressive unit stress in the concrete should not be higher than about one-sixth of its ultimate strength, and the allowable tensile unit stress in the steel should not be higher than about one-half its elastic limit. To use the author's formulas for the design of beams, two methods may be used: first, to divide his f_c and t_s by factors of safety, in order that the working unit stresses may agree with those required by the specifications; or, secondly, to multiply the given maximum bending moment by a factor of safety. While both these methods are often used, it is maintained by the writer that both are illogical, and that neither of them leads to reliable results. When a beam is to be designed to carry a given bending moment under assigned unit stresses, the design should be made from formulas in which that bending moment and those unit stresses appear, and these formulas cannot be derived except from stress-strain curves which agree with those unit stresses. The object of establishing formulas for design is to determine the proportions of beams which shall have the safe required unit stresses under the given bending moments. Such formulas cannot be expected to apply to cases of rupture, neither can formulas set up for cases of rupture be expected to give reliable results in designing. Mr. Merriman.

The author's conclusion, that the greatest economy is secured in designing a reinforced concrete beam when the cost of the steel is equal to the cost of the concrete above the steel bars, cannot be accepted by the writer. The author's investigation, starting with his Equation 10, seems to be incorrect, because it assumes the depth, d, to be a variable quantity, while his Equations 2 to 5 determine this depth for given values of the unit stresses. That is to say, the solution of these four equations, which are correct for the stress-strain curve adopted by the author, give the depth, d, of the beam, and the section area, a, of the steel per unit of breadth, in terms of the assumed unit stresses. Now, if these quantities are determined from the fundamental equations, it is certainly not in order to derive one or both of them later by supposing that they are variables, thus introducing an assumption which contradicts the fundamental conditions of equilibrium.

Another objection to the author's investigation is that he

Mr. Merriman. takes h, in Equation 6, as the constant number, 0.85, after having shown that its value ranges from 0.83 to 0.92. Now, a discussion of Equations 2 to 6 will show that this assumption fixes the steel section area at 1.13%; and, after the percentage of steel is thus fixed, it is difficult to see how it can later be made to vary with the relative costs of the steel and concrete.

There can be no doubt, however, that the proper design of a reinforced concrete beam involves the question of minimum cost. This, in the writer's opinion, is to be determined by selecting proper allowable values for the unit stresses in the steel and concrete. For the concrete the highest compressive unit stresses allowed by the specifications should be used, usually about 500 lb. per sq. in. for 1:2:4 concrete, and about 350 lb. per sq. in for 1:3:6 concrete. The tensile unit stress for the steel, however, cannot be arbitrarily assumed, but must be selected so as to make the cost of the beam a minimum, provided it be not greater than the highest value allowed in the specifications. If the tensile unit stress for the steel is taken high, the section area of the rods will be small and the beam will be deep; if it is taken low, the section area of the rods will be large and the beam will be shallow. The proper tensile unit stress to be used should be that which makes the total cost of steel and concrete a minimum.

When concrete is stressed in compression up to about 500 lb. per sq. in., the stress-strain curve is found to be closely a straight line, and hence a straight line should be used above the neutral axis in order to deduce formulas for designing. Under a small bending moment, there is also tension in the concrete on the lower side of the beam; but, under the maximum bending moment for which the beam is to be designed, these stresses are allowed to equal the ultimate tensile strength of the concrete, so that hair cracks occur. The entire tensile resistance of the concrete below the neutral axis, however, cannot be entirely overcome, but tension exists for a short distance below that axis, and should be taken into account in deriving formulas for the design of beams. The stress-strain curve for these tensile stresses is known to differ somewhat from a straight line, but, on account of the small area under tension, it may be considered as straight for a certain distance.

For 1:2:4 concrete the allowable compressive stress is about 500 lb. per sq. in., and the ultimate tensile strength about 250 lb. per sq. in. For 1:3:6 concrete the allowable compressive stress is about 350 lb. per sq. in., and the ultimate tensile strength about 175 lb. per sq. in. Hence the ultimate tensile strength of concrete is about one-half of the allowable compressive stress on the upper surface of the beam. Let C be the compressive unit stress allowed by the specifications for the upper surface of the beam, and n be the distance of the neutral axis below that surface; then, under a

straight-line law, the greatest tensile unit stress in the concrete will be $\frac{1}{2}C$ at the distance, $\frac{1}{2}n$, below the neutral axis; but, on account of the known deviation of the stress-strain curve in tension from a straight line, it will be best to apply that law only as far as $0.3n$ below the neutral axis, thus making $0.3C$ the greatest tensile unit stress, and neglecting all tensile resistance lower than $0.3\,n$ from the axis. Fig. 32 shows, then, the internal stresses proper for consideration in deriving formulas for the design of reinforced concrete beams. Mr. Merriman.

The equations of equilibrium for a rectangular beam of breadth, b, under a given bending moment, M, are now readily written. Let A be the section area of the steel rods, the center of which is at the distance, d, below the top of the beam, and let S be the tensile unit stress in these rods.

The first condition is that the sum of all the horizontal tensile stresses shall equal the sum of all the horizontal compressive stresses, whence,

$$A\ S + \tfrac{1}{2}\ b\ (0.3\ n)\ (0.3\ C) = \tfrac{1}{2}\ b\ n\ C,$$

or $$A\ S = 0.455\ b\ n\ C.$$

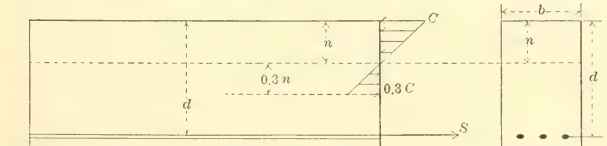

FIG. 32.

The second condition is that the resisting moment of all these stresses shall equal the bending moment. Taking the center of moments at the neutral axis, this condition gives

$$A\ S\ (d - n) + 0.045\ b\ n\ C\ (0.2\ n) + \tfrac{1}{2}\ b\ n\ C\ (\tfrac{2}{3}\ n) = M.$$

To these two conditions of statics must be added another which states the experimental fact that changes of length in horizontal lines on the side of the beam are proportional to their distances from the neutral surface. Let E_c be the modulus of elasticity of the concrete, and E_s that of the steel; then the shortening of a unit length of the upper surface of the beam is $\frac{C}{E_c}$, and the elongation of a unit length of the steel is $\frac{S}{E_s}$, and these are proportional to the distances, n and $d - n$. Accordingly,

$$\frac{C}{n\ E_c} = \frac{S}{(d - n)\ E_s}, \text{ or } \frac{d - n}{n} = \frac{S\ E_c}{C\ E_s}$$

is the third condition.

Mr. Merriman.

When a beam is to be designed, there are given its load and span, which determine the bending moment, M, the breadth, b, the moduli, E_c and E_s, and the allowable unit stress, C, for the concrete; usually, the allowable unit stress, S, for the steel is also assumed. Then the solution of the three equations above written will give the values of d, A, and n.

This solution furnishes the following formulas for designing beams, in which, for the sake of abbreviation, the ratio, $\frac{E_s}{E_c}$, is represented by the letter, e:

$$d^2 = \frac{2.92\,(e\,C + S)^2\,M}{(e\,C + 1.33\,S)\,e\,C^2\,b}. \quad A = \frac{0.455\,e\,C^2}{(e\,C + S)\,S}\,b\,d.$$

The first of these formulas gives the depth, d, and hence, also, the section area, $b\,d$, of the concrete above the reinforcing rods, while the second gives the section area of the steel. The determination of these two quantities constitutes the main part of the design of the beam when the breadth, b, is given.

In using these formulas to design a reinforced beam, the constant, e, is approximately known for each class of concrete. For all kinds of steel, E_s is closely 30 000 000 lb. per sq. in. For 1:2:4 concrete, E_c is about 3 000 000 lb. per sq. in., and hence e is about 10. For 1:3:6 concrete, E_c is about 2 000 000 lb. per sq. in., and hence e is about 15. The unit stress, C, should be taken as high as allowable by the specifications, in order to make the depth, d, as small as possible. As for the unit stress, S, it is also often customary to take the highest allowable value for steel given in the specifications, but it will now be shown by a numerical example that this practice leads to uneconomical design.

Let it be required to design a reinforced beam of 1:3:6 concrete, for which $\frac{E_s}{E_c} = e = 15$, the highest allowable unit stresses to be 350 lb. per sq. in. for the concrete, and 17 500 lb. per sq. in. for the steel. Let the breadth of the beam be 12 in.; the span, 14 ft.; and the uniform load, 300 lb. per lin. ft., including the weight of the beam. Hence, the maximum bending moment, M, is 88 200 lb-in. Using these data, Table 6 gives the depth, d, computed from the first of the above formulas for five different values of S, after which the corresponding section areas, $b\,d$ and A, are found. Each of these beams has the strength required by the specifications to carry the given bending moment with the assigned degree of security, but their costs are different. If the cost of the steel is 60 times that of the concrete, then the sums, $b\,d + 60\,A$ will be proportional to the costs for the five cases, and it thus appears that the selection of 17 500 lb. per sq. in., as the tensile working stress in the steel, produces dimensions which make the cost 3.1% more than when the

stress is taken at 12 500 lb. per sq. in. Hence it is plain that the selection of the value of S is a matter of some importance. Mr. Merriman.

TABLE 6.

S, in pounds per square inch.	d, in inches.	$b\,d$, in square inches.	A, in square inches.	$b\,d + 60\,A$.	Relative cost.
7 500	11.2	134.4	1.18	205.2	107.2
10 000	12.1	145.2	0.80	193.2	101.0
12 500	13.0	156.0	0.59	191.4	100.0
15 000	13.8	165.6	0.46	193.2	101.0
17 500	14.6	175.2	0.37	197.4	103.1

Since both $b\,d$ and A may be expressed in terms of S, it follows that the value of S which renders the total cost a minimum is a problem of pure mathematics. Let p be the ratio of the cost of one cubic unit of steel to that of one cubic unit of concrete; then the value of S is to be obtained which renders $b\,d + p\,A$ a minimum, since $b\,d + p\,A$ is proportional to the total cost of that part of the beam above the center of the steel rods. For the sake of abbreviation, let r represent the ratio, $\frac{S}{C}$, or $S = r\,C$, so that S is known as soon as r has been determined. Then the values of $b\,d$ and A are:

$$b\,d = \frac{e + r}{\sqrt{e + 1.33\,r}}\sqrt{\frac{2.92\,M\,b}{e\,C}}$$

$$A = \frac{0.455\,e}{r\sqrt{e + 1.33\,r}}\sqrt{\frac{2.92\,M\,b}{e\,C}}.$$

Multiplying the expression for A by p, differentiating the sum, $b\,d + p\,A$, with respect to r, equating the derivative to zero, and solving for r, gives

$$r = 1.17\,\sqrt{e\,p} \qquad \text{I}$$

as the value of r which is required in order that the cost of the beam shall be a minimum. Equation I is the first formula to be used in designing a reinforced concrete beam. For example, let $e = 10$ for 1:2:4 concrete, and $p = 60$; that is, let the cost of the steel per cubic unit be 60 times that of the concrete; then $r = 28.7$, or the unit stress, S, for the steel, must be 28.7 times as great as the unit stress, C, for the concrete.

After the ratio, r, has been ascertained from Equation I, the

Mr. Merriman.

distance of the rods below the top of the beam is to be computed from

$$d = u\sqrt{\frac{M}{b\,C}}, \text{ in which } u = \frac{1.71\,(e + r)}{\sqrt{e\,(e + 1.33\,r)}} \quad \text{II}$$

and then the section area of the steel is found by

$$A = v\,b\,d, \text{ in which } v = \frac{0.455\,e}{r\,(e + r)} \quad \text{III}$$

These three formulas enable the design of a reinforced concrete beam to be made which will carry the given bending moment, M, with the required degree of security, and also be more economical than one of any other dimensions. If the question of economy is not considered, Equations II and III will furnish values of d and A for any assumed unit stresses, C and S, the ratio, r, to be used being the numerical value of $\frac{S}{C}$.

The distance of the neutral axis below the top of the beam can also be stated in terms of r, and also the ratio of the cost of the steel to that of the concrete, thus

$$\frac{n}{d} = \frac{e}{e + r} \quad \text{IV}$$

and

$$\frac{p\,A}{b\,d} = p\,v \quad \text{V}$$

from which the neutral axis can be located, and the cost of the steel relative to that of the concrete computed.

Tables 7 and 8 show the relations between the different quantities more clearly than can be shown by formulas. These tables have been computed for seven values of p, ranging from 30 to 90, this quantity, p, being the ratio of the costs per cubic unit of steel and concrete. Column 2 shows the values of r which must be used in order that the beam shall be of minimum cost, this ratio, r, being computed from Equation I, and being the ratio of the tensile unit stress, S, in the steel to the given compressive unit stress, C, on the upper surface of the concrete. Column 8 of Table 7 (for 1:2:4 concrete) gives values of S in pounds per square inch when C is taken as 500 lb. per sq. in., and Column 8 of Table 8 (for 1:3:6 concrete) gives values of S when C is taken as 350 lb. per sq. in.; these columns show that the unit stresses for the steel should decrease as steel becomes cheaper with respect to concrete.

Column 3 in Tables 7 and 8 gives values of u to be used in computing the depth of the rods below the top of the beam from Equation II. Column 4 gives the values of v for computing the section area of the steel from Equation III, and these numbers multiplied by 100 give the percentage of section area of the steel with respect to the concrete section area, $b\,d$; these columns show

that the percentages of steel section to be used should increase as p decreases, and also that 1:2:4 concrete requires a slightly larger percentage of steel than 1:3:6 concrete. Column 5 shows the position of the neutral axis when the beam is stressed under the given bending moment; on the average, this axis is about 26% of the depth below the top for 1:2:6 concrete, and a little lower for 1:3:6 concrete. Mr. Merriman

Column 6 gives the ratio of the cost of the steel to that of the concrete, as found from Equation V, and this is seen to be not far from 25 per cent. Column 7, headed "Approximate relative costs," contains numbers which apply to reinforced concrete beams when properly designed by the method here presented. In computing these numbers, the allowable compressive unit stress, C, has been taken at 500 lb. per sq. in. for 1:2:4 concrete and at 350 lb. per sq. in. for 1:3:6 concrete, and no allowance for difference in cost between these two classes of concrete or for the extra concrete below the reinforcing rods has been made. Under this supposition, the cost of reinforced beams is about 10 or 11% higher when the lower grade of concrete is used.

TABLE 7.—FOR 1:2:4 CONCRETE. $\frac{E_c}{E_s} = e = 10.$

(1) Ratio, p	(2) r	(3) u	(4) v	(5) $\frac{n}{d}$	(6) $\frac{p\,A}{b\,d}$	(7) Approximate relative costs.	(8) Unit Stress, S, for C=500.
90	35	3.2	0.0029	0.22	0.26	100	17 500
80	33	3.2	0.0032	0.23	0.26	98	16 500
70	31	3.1	0.0036	0.24	0.25	95	15 500
60	29	3.0	0.0041	0.26	0.25	92	14 500
50	26	2.9	0.0049	0.28	0.24	89	13 000
40	23	2.8	0.0060	0.30	0.24	85	11 500
30	20	2.7	0.0076	0.33	0.23	81	10 000

TABLE 8.—FOR 1:3:6 CONCRETE. $\frac{E_c}{E_c} = e = 15.$

(1) Ratio, p	(2) r	(3) u	(4) v	(5) $\frac{n}{d}$	(6) $\frac{p\,A}{b\,d}$	(7) Approximate relative costs.	(8) Unit Stress, S, for C=350.
90	43	3.0	0.0027	0.26	0.24	110	15 100
80	41	3.0	0.0030	0.27	0.24	108	14 400
70	38	2.9	0.0034	0.28	0.24	105	13 300
60	35	2.8	0.0039	0.29	0.23	102	12 300
50	32	2.7	0.0045	0.32	0.22	99	11 200
40	29	2.6	0.0055	0.34	0.22	95	10 200
30	25	2.5	0.0069	0.38	0.21	90	8 800

Mr. Merriman.

In Equation II, for computing the depth, d, the letter, C, appears, so that this formula may be used for any specified compressive unit stress. This formula, of course, is only one of many that may be deduced for different assumptions regarding the tensile stresses below the neutral surface, each assumption giving a different expression for u. While the common assumption, that the concrete below the neutral surface offers no tensile resistance, leads to somewhat different formulas for u and v, the numerical results obtained from them do not differ materially from those above given, as far as the values of u and v are concerned, although they give the position of the neutral axis somewhat higher. The writer has also worked out formulas and tables under the assumption that the tensile stresses in the concrete extend to the distance, $\frac{1}{2}n$, below the neutral axis, and finds that this gives the depths of beams and section areas of reinforcement about 3% greater than when no tension in the concrete is considered. Hence, neglect of the tensile resistances in the concrete is not on the side of safety when beams are to be designed.

The writer's conclusions regarding the proper design of reinforced concrete beams are as follows:

1.—Formulas deduced from stress-strain curves of concrete tested to rupture in compression are irrational and unreliable for the design of beams.

2.—After the three fundamental equations have been written for any assumed stress-strain line, no further assumptions regarding the neutral axis or the lever arms of forces can be made without introducing contradictions which render the resulting formulas erroneous.

3.—When the unit stresses, C and S, are assumed, the three fundamental equations determine the depth of the beam and the section area of the steel.

4.—The practice of using for structural steel a tensile stress as high as one-half the elastic limit leads to uneconomical design, unless the cost of steel per cubic unit is greater than about 90 times the cost of 1:2:4 concrete, or greater than about 100 times the cost of 1:3:6 concrete.

5. The unit stress, S, to be used for the steel rods, should be such that the cost of the reinforced beam shall be a minimum, and this value may be ascertained from Equation I.

6.—When no precise information is at hand, regarding the relative costs of steel and concrete, the value of r, in Equations II and IV, may be taken at about 31 for 1 : 2 : 4 concrete and at about 35 for 1 : 3 : 6 concrete. This gives the section areas of the steel as 0.36 and 0.39% for these two classes of concrete.

7.—Steel with a high elastic limit should not be used for rein-

forcing rods if its price per pound is higher than that of structural medium steel. Mr. Merriman.

8.—Some formulas and tables in use require percentages of steel section area to an extent which is not only unnecessary, but wasteful and extravagant. The highest steel section area which may be used for economical design is 0.75% of the concrete area above the rods, and this is only allowable when the cost of steel per cubic unit is as low as 30 times that of the concrete.

9.—For economical design, the cost of the steel is about 25% of the cost of that part of the concrete which lies above the centers of the reinforcing rods. The use of steel to an extent which renders its cost equal to the cost of the concrete section, $b\ d$, leads to uneconomical design.

10.—A high-class concrete, for which the modulus of elasticity is about 3 000 000 and the allowable compressive stress 500 lb. per sq. in., is more economical for reinforced beams than a concrete having a modulus of 2 000 000 and an allowable compressive stress of 350 lb. per sq. in., unless the cost of the latter concrete is at least 10% less than that of the former.

A. H. PERKINS, ASSOC. M. AM. SOC. C. E. (by letter).—That the moment equation for a reinforced concrete beam, as determined by any of the rational formulas for a given value of f_s, approximates very closely to a straight line, is a familiar fact to engineers working with curves such as those published in *Engineering News* by Mr. Schaub as long ago as April 30th, 1903. There the moment curve is plotted with $\frac{a}{d}$ as ordinates and $\frac{M}{b\,d^2}$ as abscissas. That the curve with $\frac{a}{d}$ constant and f_s and $\frac{M}{b\,d^2}$ the varying elements also approximates a straight line, is new, and the resulting general equation is extremely simple. There is a general feeling among engineers, however, that what is wanted is not so much accurate formulas, although they are desirable, as accurate constants for use in formulas. Knowledge of proper values of E_c could hardly be in a more chaotic condition, and an extension of the knowledge of the proper percentages of reinforcement to be used with various mixtures would be received with pleasure by most engineers. However, even though the cart has been obtained before the horse, the possession of the cart is cause for congratulation. Mr. Perkins.

That 0.8 represents the "reliability factor" of the strength of concrete, as compared with steel at its elastic limit, will be questioned by many engineers. The elastic limit of concrete is not more than three-fourths of its ultimate strength for the leaner mixtures, such as 1:3:6. This is shown clearly by the data presented by

Mr. Perkins. Professor Hatt* and by the Talbot experiments. With the richer mixtures, more common in reinforced construction, the elastic limit is probably a somewhat greater percentage of the ultimate strength. In addition, there is the variation in the strength of the same grade of concrete. It would seem, therefore, that it would have been better to have left the factor, 0.8, out of the equations, permitting the user to suit himself or the conditions in choosing the corresponding factor.

The writer observes that the author finds by his analysis the same position for the centroid of pressure that was obtained by Professor Talbot by analysis ($= \frac{7}{11}$ of y, above the neutral axis) at the time he reported his now classical set of experiments.

However completely a beam may be reinforced, it will fail as soon as the elastic limit of the metal is passed unless the percentage of reinforcement is below that which develops the full strength of the concrete at the elastic limit of the metal, for the reason that when the elastic limit of the metal is passed the neutral axis rises rapidly, and the unit stress in the upper fiber of the concrete increases. Hence, designing a beam that will not go to pieces when the elastic limit of the metal is passed, involves under-reinforcement. Web reinforcement is necessary, of course, under certain conditions, and perhaps desirable under nearly all conditions; however, it will not produce impossibilities, and should not be expected to produce a beam that will not go to pieces soon after the elastic limit of the metal is reached, if the percentages of reinforcement recommended by the author are used with ordinary mixtures. The elongation of 30 000-lb. elastic-limit steel at the elastic limit will be 0.1%, and between the elastic limit and the ultimate it will be about 25 per cent. For steel of 50 000-lb. elastic limit, the elongation at the elastic limit is 0.17% and at the ultimate 15 per cent. Now throw the author's Equation 4 into the form

$$k = \frac{n f_c}{n f_c + f_s}, \quad \text{where} \quad k = \frac{y_1}{d}, \quad \text{and} \quad n = \frac{E_s}{E_c}.$$

This is true for any shape of the stress-strain curve within the elastic limits of the materials, and may be used to find k at the ultimate by putting for f_s, 250 times f_s for steel of 30 000-lb. elastic limit, or 88 for steel of 50 000-lb. elastic limit (f_s remaining the stress at the elastic limit). It is evident that these values will give such attenuated values for k that the amount of reinforcement permissible would be "the ghost of a departed quantity."

This leads directly to the vexed question of factors of safety. If the concrete be reinforced up to the point where the steel reaches the elastic limit when the upper fiber of the concrete is at its ultimate, then the factor of safety of the concrete is only half that

* *Transactions*, Am. Soc. C. E., Vol. LIV, Part E, pp. 587 *et seq.*

TABLE 9.

Mr. Perkins.

f_s.	k.	$\frac{a}{d}$.	F.	$f_s(6M_c)$.	$\frac{E_s}{E_c}$.	Mixture.	$\frac{M}{bd^2}$.
30 000	0.4	0.0152	2 000	3 000	10	1 : 2 : 4	408
	0.333	0.0095	1 500	2 500	10	1 : 3 : 6	251
40 000	0.333	0.0095	2 000	3 000	10	1 : 2 : 4	339
	0.273	0.0058	1 500	2 500	10	1 : 3 : 6	211
50 000	0.286	0.0065	2 000	3 000	10	1 : 2 : 4	293
	0.231	0.0040	1 500	2 500	10	1 : 3 : 6	181
60 000	0.250	0.0048	2 000	3 000	10	1 : 2 : 4	260
	0.200	0.0029	1 500	2 500	10	1 : 3 : 6	159

of the steel, a condition the reverse of what it should be, and that is manifestly a waste of metal. On the other hand, if $\frac{a}{d}$ be taken below the above-mentioned value, the beam goes to pieces before the full ultimate stress in the concrete is used. In other words, there is no way to utilize the factor of safety of 2 that steel has between the elastic limit and the ultimate strength. Of the two horns of the dilemma, it is clear to the writer that stressing the concrete below its ultimate is by far the shorter horn, and the better engineering. The writer, therefore, would use, in the author's formulas, the values of $\frac{a}{d}$ and F shown in Table 9 modified by leaving out the factor, 0.8.

Perhaps a larger value of $\frac{E_s}{E_c}$, say 12, should be taken for the 1:3:6 mixture. This would make the resulting values of $\frac{a}{d}$ slightly larger. To the $\frac{M}{b\,d^2}$ in Table 9 the writer would apply a factor of safety of at least $2\frac{1}{2}$ for dead loads.

Langdon Pearse, Jun. Am. Soc. C. E. (by letter).—The writer has read Captain Sewell's interesting paper and the discussion thereon with much pleasure. He would like to add a few notes on the stress-strain curve of concrete in compression, based on tests made for the Boston Elevated Railway.* Mr. Pearse.

These notes, made in connection with a thesis at the Massachusetts Institute of Technology in 1902, refer only to the 1 : 2 : 4 mixture. Plots of the stresses and strains were made on logarithmic paper, first the inelastic strains, that is, the strains produced by the first loadings, and second the elastic strains, that is the first strain minus the permanent set. Logarithmic plotting paper was used because of the well-known property that the logarithms of points on a parabola lie in a straight line.

* "Tests of Metals," U. S. War Dept., 1899.

TABLE 10.—INELASTIC STRESS-STRAIN CURVE FOR 1 : 2 : 4 CONCRETE.

Cement.	Time of Set.	y_1	x_1	y_2	x_2	n	p	y_3	x_3	y_4	x_4	n_1	p_1	Breaking strength, in pounds per square inch.	Remarks.
Alpha.....	7 d.	190	0.0001	300	0.0006	3.195	2.043×10^{13}	300	0.00062	700	0.0044	2.313	8.648×10^{8}	932	
Alpha.....	1 mo.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2 590	Points too scattered to draw curves.
Alpha.....	3 mo.	500	0.0001	2 300	0.001	1.484	1.011×10^{8}	.....	.....	.....	.....	.....	.....	3 226	
Alpha.....	6 mo.	430	0.0001	2 400	0.001	1.285	2.207×10^{7}	.....	.....	.....	.....	.....	.....	4 847	
Atlas.....	8 d.	350	0.0001	1 000	0.001	2.193	3.802×10^{9}	1 000	0.001	1 600	0.0051	3.469	5.22×10^{13}	1 695	
Atlas.....	1 mo.	420	0.0001	1 800	0.001	1.582	1.414×10^{8}	.....	.....	.....	.....	.....	.....	2 373	Curve breaks just below 1800.
Atlas.....	3 mo.	450	0.0001	2 100	0.001	1.495	9.236×10^{7}	2 100	0.001	2 900	0.004	4.294	1.845×10^{13}	2 918	
Atlas.....	6 mo.	400	0.0001	2 060	0.001	1.405	4.57×10^{7}	2 400	0.0012	2 900	0.004	6.362	2.667×10^{13}	4 000	
Germania.	7 d.	370	0.0001	1 240	0.001	1.901	7.759×10^{8}	.....	.....	.....	.....	.....	.....	2 400	
Germania.	1 mo.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2 933	Micrometer broke.
Germania.	3 mo.	500	0.0001	2 000	0.001	1.661	3.04×10^{8}	.....	.....	.....	.....	.....	.....	3 157	
Germania.	6 mo.	460	0.0001	1 940	0.001	1.600	1.82×10^{8}	1 940	0.001	3 000	0.0032	2.668	5.93×10^{13}	3 309	
Alsen.....	10 d.	335	0.0001	1 180	0.001	1.828	4.145×10^{8}	1 180	0.001	1 840	0.0043	3.283	1.22×10^{13}	1 862	
Alsen.....	1 mo.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2 373	Curve made of more than 2 parabolas.
Alsen.....	3 mo.	355	0.0001	1 560	0.001	1.555	9.262×10^{7}	.....	.....	.....	.....	.....	.....	2 600	Curve drops near end.
Alsen.....	6 mo.	420	0.0001	2 260	0.001	1.368	3.881×10^{7}	.....	.....	.....	.....	.....	.....	3 809	Curve drops.
Saylor's...	9 d.	184	0.0001	700	0.001	1.723	8.00×10^{7}	.....	.....	.....	.....	.....	.....	1 944	
Saylor's...	1 mo.	420	0.0001	1 300	0.001	2.081	3.023×10^{10}	1 300	0.001	2 100	0.006	3.736	4.296×10^{13}	2 119	
Saylor's...	3 mo.	430	0.0001	1 800	0.001	1.635	2.093×10^{8}	1 800	0.001	2 600	0.004	3.770	1.866×10^{13}	2 778	
Saylor's...	6 mo.	500	0.0001	2 300	0.001	1.509	1.181×10^{8}	2 400	0.0011	4 200	0.0045	2.584	5.10×10^{13}	3 959	

Mr. Pearse.

Mr. Pearse.

An examination of the inelastic strain plots showed that in few cases could one straight line be passed through all the points plotted, whereas in many cases two straight lines at a considerable angle, and, in one or two cases, three straight lines could. This would mean that two or more parabolas might represent the curves. The elastic strains followed the straight lines more closely, so that one line was fairly representative for each set of observations.

In both cases the constants were derived for the range of stress from 300 lb. per sq. in. up to the ultimate strength, unless otherwise noted, assuming that the probable curve was $y^n = p\ x$,

where y = the stress, in pounds per square inch;

x = the change of length or strain, in inches per inch.

n and p are the constants to be derived.

The values used in the computations are given in Tables 10 and 11. The columns headed y_1 and x_1 give the low point, and y_2 and x_2 the high point from which the constants, n and p, were calculated. The columns headed y_3 and x_3, and y_4 and x_4, give the points from which the values of n_1 and p_1 were determined. Tables 10 and 11, respectively, give the inelastic and elastic constants.

Using $y^n = p\ x$ as the stress-strain equation, according to the definition of the modulus of elasticity, E, as the ratio of the change of stress to the change of strain, then

$$E = \frac{d\ y}{d\ x} = \frac{p}{n\ y^{n-1}}.$$

TABLE 11.—ELASTIC STRESS-STRAIN CURVE FOR 1 : 2 : 4 CONCRETE.

Cement.	Time of set.	y_1	x_1	y_2	x_2	n	p	Remarks.
Alpha.....	7 d.	234	0.0001	820	0.0008	1.667	91 650 000	
Alpha.....	1 mo.	.....	.....	.....	.....	.....	.....	
Alpha.....	3 mo.	564	0.0001	2 840	0.0010	1.425	83 040 000	
Alpha.....	6 mo.	422	0.0001	3 200	0.0010	1.130	9 310 000	
Atlas.....	8 d.	395	0.0001	1 520	0.0010	1.708	273 300 000	
Atlas.....	1 mo.	425	0.0001	1 690	0.0010	1.668	242 000 000	
Atlas.....	3 mo.	515	0.0001	2 390	0.0010	1.500	116 800 000	
Atlas.....	6 mo.	442	0.0001	2 640	0.0010	1.289	25 580 000	
Germania...	7 d.	369	0.0001	1 820	0.0010	1.443	50 540 000	
Germania...	1 mo.	.....	.....	.....	.....	.....	.....	Micrometer broke.
Germania...	3 mo.	468	0.0001	2 600	0.0010	1.343	38 540 000	
Germania...	6 mo.	525	0.0001	1 840	0.0006	1.428	76 900 000	Curve flattens at top.
Alsen.....	10 d.	390	0.0001	1 800	0.0010	1.505	79 650 000	
Alsen.....	1 mo.	420	0.0001	2 090	0.0010	1.435	58 100 000	
Alsen.....	3 mo.	420	0.0001	1 630	0.0007	1.435	58 130 000	Curve flattens above this.
Alsen.....	6 mo.	570	0.0001	2 430	0.0008	1.434	89 600 000	
Saylor's...	9 d.	298	0.0001	1 560	0.0010	1.359	27 650 000	
Saylor's....	1 mo.	450	0.0001	1 800	0.0010	1.661	255 300 000	
Saylor's....	3 mo.	455	0.0001	2 190	0.0010	1.465	78 560 000	Curve flattens at top.
Saylor's....	6 mo.	580	0.0001	1 900	0.0005	1.356	56 020 000	

Mr. Pearse.

This is the slope of the curve at the point, x_1 y. Now if $y = 0$, $E = \infty$. This is clearly untrue, although for small values of stress the real strain is not known. It is probable that concrete is elastic, in the ordinary sense, up to 200 or 300 lb. per sq. in., and even up to 500 lb. per sq. in., in some cases, though permanent set is usually measured below that stress. The correct expression for the stress-strain curve in compression would seem to be a parabola with the vertex at zero, but with its axes slightly revolved so that the slope is finite at the origin, or else a combination of parabola and straight line—the straight line extending to the point at which permanent set is noted, the parabola from there to the point of ultimate strength.

For all practical purposes, the writer favors the formulas based on a straight-line stress-strain curve, using constants based on working values of the fiber stresses in steel and concrete, neglecting the concrete in tension.

Mr. Wing.

C. B. WING, ASSOC. M. AM. SOC. C. E. (by letter).—In a recent paper before the Canadian Society of Civil Engineers, Henry Goldmark, M. Am. Soc. C. E., has stated that most of the formulas proposed for the design of reinforced concrete are based on the common theory of flexure of homogeneous materials, with modifications due to the composite nature of the beam and physical properties of concrete differing from those of steel.

A more accurate statement would be that proposed formulas may be divided into two classes, one class for which Mr. Goldmark's statement is true, and another class in which the modifications and assumptions are of such a character that all semblance to the ordinary theory of flexure is lost, and the resulting formulas are purely empirical.

The formula proposed by the author is of this latter class; and, for low percentages of reinforcement, with material of low elastic limit, will be found to give ultimate strengths agreeing closely with the results of tests.

The range of application of the formula, however, is limited, as will be shown later, and in inexperienced hands may give extremely dangerous results.

The stress-strain diagram of a reinforced concrete beam shows two critical points: the point of failure of the concrete in tension, and the elastic limit of the steel.

These points may be compared with the elastic limit and the point of rupture of the stress-strain diagram of steel in tension, with the difference that, in the case of steel, stresses beyond the elastic limit cause permanent deformation, while, in the case of reinforced concrete, stresses beyond the tensile strength of the concrete only lead to the opening up of cracks which are closed on the re-

moval of the load if the elastic limit of the metal has not been ex- Mr. Wing.
ceeded.

The whole question of proper methods of designing reinforced concrete would seem to hinge on this one point, that is, to what extent it is safe to allow tensile cracks to form in reinforced concrete beams.

This point can only be settled satisfactorily by tests for the effect of corrosion and repeated stress on reinforced concrete beams in which such cracks have formed.

Until such tests have been made, conservative design would require that the maximum stresses in the outer fiber of reinforced concrete beams be kept within the limits of the ultimate tensile strength of the concrete, say 300 lb. per sq. in. for a fair quality of concrete.

If this principle is accepted, the best type of formula to use is one based on the ordinary theory of flexure.

At present the design of beams by such formulas is comparable to the condition that would exist if all tables of moments of inertia and properties of steel beams were to be destroyed.

However, by adopting a proper notation, and preparing tables, it is possible to solve theoretical composite beam formulas with the same ease that solutions of formulas for homogeneous beams are obtained.

Such theoretical formulas can be shown to give results agreeing closely with the results of tests, both at the point of failure of the concrete in tension and at the point at which the elastic limit of the steel is reached.

This being the case, there would seem to be no justification for adopting a formula theoretically open to criticism, of limited application, and which, in inexperienced hands, would give dangerous results.

Table 12 has been prepared in order to show the difference in designs obtained by using the author's formulas with $f_s = 16\,000$ lb. per sq. in., and by using a formula based on the ordinary theory of flexure limiting the tensile stress in the concrete to 300 lb. per sq. in.

The beams in Table 12 have been calculated as having a resisting moment of 120 000 in-lb. The beams calculated by the ordinary flexure formulas are square, with the center of the steel reinforcement 1 in. from the lower surface of the concrete. The beams calculated by the author's formula are square, above the center of the steel, and have 1 in. added to the depth to provide a protective coating for the steel.

The cost of concrete was assumed at 20 cents per cu. ft., and the cost of steel at 3 cents per lb.

The author states that "the actual working stress in the concrete would seem to be of secondary importance as long as the factor of

Mr. Wing. safety is assured." It is difficult to understand how the factor of safety is assured when beams designed by the author's formula show probable values of the compressive stress in the concrete as given in Table 12. These values have been calculated by the ordinary theory of flexure, neglecting the tension in the concrete, and may be higher than the stresses actually existing, but beams calculated by such formulas are on the side of safety.

The author, in justification of an empirical formula, speaks of the plate girder as a case in which the designer departs from the ordinary theory of flexure, but fails to state that such departure leads to the design of heavier beams than would be required by the ordinary theory of flexure.

It is to be regretted that the author's proposed formula and other similar formulas do not in all cases in like manner give results departing from the ordinary theory of flexure on the side of safety. It is safe to say that, if such were the case, technical literature would be but little burdened with discussions of the true form of the stress-strain curve for concrete in compression, and the old-fashioned Hooke's Law would be considered near enough for practical purposes.

The portion of the paper devoted to the determination of the economic percentage of reinforcement depends entirely upon the acceptance of the proposed formula. If that is not accepted, the whole argument is without foundation, and that the acceptance of this formula may lead to dangerous designs is indicated by the results given in Table 12, which show that beams designed by the ordinary flexure formulas will increase in cost with the percentage of reinforcement.

For economy, the percentage of reinforcement, therefore, should be as small as good practice will warrant, and is not a subject for theoretical determination.

TABLE 12.

Ordinary Flexure Formula.					Author's Formula.		
Percentage of reinforcement.	Side of square beam, in inches.	Cost per foot, in cents.	Probable maximum stresses per square inch, if concrete fails in tension.		Depth of beam ; width 1 in. less.	Cost per foot, in cents.	Probable compression in concrete per square inch.
			Tension in steel.	Compression in concrete.			
$\frac{1}{2}$	12.8	31.0	14 300	530	12.4	26.1	640
$\frac{3}{4}$	12.6	33.6	10 000	490	11.0	22.8	800
1.0	12.3	36.1	8 100	470	10.1	21.1	1 040
$1\frac{1}{4}$	12.1	38.3	6 800	460	9.4	19.8	1 130
$1\frac{1}{2}$	11.9	40.5	6 000	460	8.9	19.2	1 280
$1\frac{3}{4}$	11.7	42.6	5 500	460	8.5	18.9	1 440
2.0	11.5	44.7	5 000	460	8.2	18.6	1 560

Mr. Cain. WILLIAM CAIN, M. AM. SOC. C. E. (by letter).—In the case of wooden or iron beams, the earlier theory for rupture was first used in practice, and, later, a more correct theory for working stresses within the elastic limit. It seems that the theoretical treatment of concrete beams, reinforced or not, will have to pass through the same stages. To reach this desired goal, however, additional experiments will have to be made, especially on prisms of plain concrete, for repeated stresses, so as to eliminate, in the computations, the permanent set, and give an elastic modulus of elasticity which does not alter, with repeated loads, within assigned working limits.

Thus, suppose a concrete prism compressed by an axial load which gives a unit stress, c, and a shortening of the prism per unit of length, ϵ, then $\frac{c}{\epsilon}$ is the modulus of elasticity for this loading applied once. On removing the load, a permanent set is observed. By applying and removing the same load a number of times, the permanent set is steadily increased, and the modulus, computed as above (e including the permanent set), varies although c remains constant. Therefore it is unfit for accurate computation. However, by repeating the loading and unloading a sufficient number of times, the permanent set, per unit of length, finally becomes a constant, k; so that on subjecting the prism any number of times thereafter to the same loading as at first, the shortening, e', per unit of length, over and above the permanent k, disappears when the load is removed. It is thus an elastic deformation. The total shortening of the prism, per unit of length, elastic and inelastic, is thus, $k + e'$.

The elastic modulus of elasticity of the concrete in compression for repeated loads within an assumed working stress, c, may then be defined as $\frac{c}{e'}$. This is a constant, and is suitable for use in accurate computation for concrete, of the same kind and the same age, for repeated stresses from 0 to c.

Unfortunately, however, it is found that this so-called elastic modulus decreases as c increases, as shown by the results in Table 13*, as given by Professor Bach. The tests were made at from 3 to 4 months after moulding, and thus only apply for such ages of concrete.

The values of E_c on the last two lines are for shingle concrete. Doubtless future observations will modify these results materially, but, granting their correctness for the materials used and the age of the specimens stated, it is observed that as c increases, the values of E_c decrease most rapidly at first until, beyond $c = 500$, the de-

* See more complete table in "Reinforced Concrete," by Charles F. Marsh, p. 214, from which these results were taken.

Mr. Cain. crease is slight and nearly uniform. The "stress-strain" curve, consequently, has a greater curvature for the small values of c than for values from 500 upward, and it only approaches a straight line for values of c from, say, 500 upward. It is unfortunate that this relation was not reversed, for it is highly desirable to use the straight line from $c = 100$ up to $c = 500$ in the computations for beams, where the maximum fiber compressive strength on the concrete is limited to 500 lb. per sq. in.

TABLE 13—Elastic Moduli for Stone Concrete.

(Multiply the values of E_c in this table by 1000).

Cement.	Sand.	Broken Stone.	Values of c, in pounds per square inch.									
			114	228	342	456	570	684	798	912	1 026	1 140
			Values of Ec, in pounds per square inch.									
1.....	2.5	5	466	417	391	374	359	349	341	335	328	323
1.....	3	6	387	347	326	311	299	291	283	277	273	268
1.....	5	10	344	300	276	262	249	240	233	227	222	220
1.....	2.5	5	317	287	271	262	252	246	242	237	233	230
1.....	5	10	222	198	185	178	170	166	162	158	157	156

However, the average modulus between $c = 100$ and $c = 500$ can be used in connection with the straight-line stress-strain diagram, without a large error in computing the maximum fiber stress in the concrete, and practically with none at all for the stress in the steel.

Thus, for a working stress in the 1-5-10 concrete of about 500 lb. per sq. in., take $E_c = 3\,000\,000$, then $m = \frac{E_s}{E_c} = \frac{30}{3} = 10$, and use the formulas pertaining to the straight-line stress-strain diagram, or preferably a drawing similar to the admirable one given by Professor French (Fig. 28), which controls "automatically" the relation of f to c. It would prove desirable, of course, to prepare other sheets for varying values of m. Professor Merriman, in his discussion, has given a more accurate solution, in that it considers a portion of the concrete below the neutral axis as acting in tension.

Of course, in both solutions, unless the elastic modulus is used, the work is irrational and misleading, and this elastic modulus must be found as c varies from 0 to the working limit—whether 350 or 500 lb. per sq. in.

Mr. Cain. These straight-line (stress-strain) solutions are readily used to ascertain comparative costs and thus lead to economic design. Considering the widely varying values of E_c by different experimenters under the same conditions, and the fact that E_c varies so widely with the age of the specimen, it seems to be useless to enter into great refinement as to the stress-strain curve; so that, when the "elastic" modulus is considered, the straight-line stress-strain diagram can be used up to $c = 500$, or even $c = 1000$, with but little error. Ultra refinement seems to be useless, when, in addition to the uncertainties about the modulus, the unknown stresses due to shrinkage and permanent set due to repeated loadings are entirely neglected.

As to the shrinkage stresses caused by the contraction of certain reinforced prisms of cement mortar while setting in air, M. Considére found that the reinforcement used suffered a compressive stress of 2 845 lb. per sq. in., and the mortar a mean tensile stress of 155 lb. per sq. in. In neat cement reinforced prisms, the results were 7 110 and 410 lb. per sq. in., respectively.* Presumably, in stone concrete, from the nature of the aggregates, there would be less initial tension (or shear) from setting in air. In a concrete beam reinforced on one side only, the shrinkage due to setting in air, being resisted by the steel, should cause tension in the concrete near the reinforcement, and therefore compression at the opposite face, leading to bending, though the writer is not aware of this phenomenon having been observed. If the theory is true, the stresses induced are in the line of danger. When the beam is loaded, so that cracks appear on the tension face, these initial stresses are relieved at the cracks, but not between the cracks. The steel rod is not thus subjected throughout to a uniform stress.

That initial stresses in a reinforced beam must be caused by repeated loadings also must be evident from the following considerations.

In Fig. 33, representing a portion of a beam (for simplicity supposed to be without weight), let AB be a normal section before stress. One application of the load stretches a fiber at B of unit length, an amount, BB', and shortens the fiber at A the amount, AA'. If there was no reinforcement, on releasing the load, $A'B'$ (supposed plane) moves to $C\,N\,O'$, giving permanent sets, $A\,O'$ at A, and $C\,B$ at B. But, with a rod reinforcement near the lower edge, the tension in the rod tends to diminish BC, and therefore AO', causing tension in the concrete at A and compression at B. After a number of

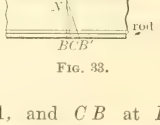

FIG. 33.

* Marsh's "Reinforced Concrete," p. 253.

Mr. Cain.

applications and releases of the same load, AA' and AO' increase, finally becoming constant, similarly with BC and BB'.

There are thus initial stresses, from repeated loadings, due to the one-sided reinforcement, which, fortunately, are on the side of safety, being tensile at A and compressive at B.

Professor Talbot performed some experiments on the "release and repetition of load," and in the "University of Illinois Bulletin" for September 1st, 1904, Fig. 42 shows the results for Beam No. 20. As Professor Talbot remarks, Fig. 42 "seems to indicate that the steel and concrete are under stress after the load is released."

It is to be hoped that experimenters will direct some of their attention to repeated loadings, for unless the elastic modulus of concrete is found, all theories within working limits give misleading results.

If it is simply desired to derive a formula that will agree with the results for a beam tested to destruction under non-repeated loads, then the inelastic modulus will suffice, and such formulas are useful. It seems to the writer that the full stress-strain curve should be used for rupture and for rupture only. If this curve is a parabola, then Professor Talbot's formulas are rational and exact, not only for rupture, but for any working stresses in the concrete; provided the tension in the concrete below the neutral axis is ignored. His formulas, however, demand an "initial" modulus, which is theoretically double the modulus at rupture. As yet, such initial moduli have not been well determined, even for non-repeated stresses. It is absurd, of course, to apply the modulus at rupture to formulas where the initial modulus is called for, or where the modulus for repeated loads within working limits is required.

It must be perceived clearly that a formula for rupture cannot be used for working limits by the use of a factor of safety, and *vice versa*. Two distinct formulas, using different moduli, are required for the two cases.

The author deserves the thanks of the profession for having started this discussion, and for his interesting contribution to it.

Captain Sewell.

JOHN S. SEWELL, M. AM. SOC. C. E.* (by letter).—After reading the discussion which has been brought out by his paper, the writer feels that its main object, which was to arouse a greater interest in certain points pertaining to the design of reinforced concrete, has been largely accomplished. He desires to express his gratitude for the kindly expressions of appreciation by so many of those taking part in the discussion.

In order to set at rest the doubts apparently existing in the minds of some, he desires to say that he claims no originality for anything fundamental in any part of the formulas discussed or

*Captain, Corps of Engineers, U. S. Army.

Captain Sewell.

proposed by himself. Even the treatment of web stresses, which is different from any he has seen elsewhere, could not be called original, in the fullest sense of the word, and it may not be so, in any sense. The question of originality, however, is entirely secondary to that of correctness. The writer, in common with many other engineers, has studied such data as he had available, and, in the light of such study and his own experience, has deduced conclusions in which he has great confidence. But it is realized that these conclusions are rather matters of opinion than thoroughly demonstrated facts, and it is hoped that the extensive series of tests about to be inaugurated under the auspices of this and other societies may cover the ground in so thorough a manner as to leave no room for further doubt or discussion.

The point raised by Mr. Jonson, in reference to taking the depth of the horizontal reinforcement below the top of the beam as a basis for the shear computation, is well taken. Referring to page 277, and to Fig. 4, the depth should be $h\ d$, instead of d. The writer had intended to correct this error, but will content himself by acknowledging the accuracy of Mr. Jonson's criticism. It should, perhaps, be further explained, that Fig. 4 and its accompanying text constituted a demonstration of the writer's method of treating the web stresses, rather than a statement of the method itself. As a matter of fact, when the writer uses diagonal web members, if a be the area of the horizontal reinforcement, the aggregate section of the web members in one-half of the beam is taken as $\frac{1}{2}\ a\ \sqrt{2}$; if vertical web members are used, their aggregate area in one-half of the beam is taken as a; neither of these expressions would be quite correct if the depth were taken as d; nor would they result from an actual determination of stresses in a multiple-intersection truss with a finite number of web systems. The greater the number of web systems, the more nearly does the aggregate of the tensile web stresses approach equality with the aggregate of the compressive stresses; and, in a solid beam, they become equal; under these conditions, the writer's expressions for the aggregate section of web members are correct. If diagonal members are used, the length of each member is equal to $d\sqrt{2}$. The total volume of the members in one-half of the beam is then equal to $a\ d$. The same expression holds for the volume of the vertical members, so that the total weight of the web reinforcement is the same in the two cases, assuming the web members to extend to the top of the beam, in either case. In any case, the number of web members necessary to make up the aggregate section is determined by dividing the aggregate by the area of one member. Their spacing is determined in much the same manner as that used for the rivet spacing for a plate girder.

Captain Sewell.

Replying to Mr. Watson: the unreasonable and illogical requirements of many building laws constitute one of the reasons for writing this paper. These laws often, on the one hand, put reinforced concrete at an unfair disadvantage, and, on the other, open up the way to very real dangers in design. The writer is also opposed to the use of high-carbon steel, or any steel with an elastic limit exceeding, say, 40 000 to 45 000 lb. per sq. in. If the modulus of elasticity could be increased with the elastic limit, the matter might be different; but, as it is, the greater strength of the steel with a high elastic limit can be utilized only by permitting deformation beyond a reasonable limit, and by permitting the neutral axis to rise so high in the beam that the economy of the greater stress in the steel is largely lost, because of the greater quantity of concrete needed to counterbalance the steel stresses. The strength of the steel with a high elastic limit is also likely to be seriously impaired in a fire, which is in itself a sufficient reason for using soft or medium steel in fire-resisting structures, not to mention the other very valid objections to high-carbon steel urged by Mr. Watson.

Vertical stirrups, merely passing under the main bars, cannot possibly transmit any tensile stress into those bars, for no force has a component at right angles to itself; nor can such stirrups afford an abutment for the diagonal compressive stresses in the concrete, except in so far as the concrete itself serves as an anchorage for them. As one cannot lift himself by his own boot straps, it is difficult to know how the stirrups described by Mr. Watson can assist in any way except by reinforcing the concrete against local failure, and thus holding adhesion up to its full value. This seems to be only a partial solution of the real problem.

Replying to Mr. Noble: the writer expects that well-conducted experiments will prove quite conclusively that percentages of reinforcement, considerably greater than those deduced from his own equations, can be safely used. The writer, in assuming his constants, tried to keep well within safe limits, and the discussion indicates that in this, at least, he was successful.

The great value of the adhesion, and of the shearing strength, in concrete is not denied; but both are subject to at least as many uncertainties as the tensile strength, and the writer prefers to let them all go into the indeterminate part of the factor of safety—especially in structures likely to be damaged by fire.

Mr. Kreuger's comments, concerning the width of flange in T-beams, touch upon a very interesting point—one which, as it is treated in such building laws as those of New York City, opens up the way to some decidedly dangerous designs. The writer's width of flange was deduced from a discussion based on the shearing strength of concrete. While he now disregards this, as far as

possible, the width of the rib, in a T-beam, results from considera- Captain Sewell.
tions affecting the transfer of the tensile stress from the steel into the compressed part of the concrete; the questions of how much stress can be distributed into the slab on either side, and how far it must go before being absorbed, are not usually easy to determine. But, the length of the span, and all other real factors, enter, indirectly, into the design of the rib, and a width three times as great as the width of the rib will usually be found quite safe and sufficient for the flange, except in the rare case where very heavy beams are spaced very closely together, with a very thin floor slab on top of them. In that case, the thickness of the flange would seem to be the correct basis for determining its width. The writer sees no objection to making it the basis in all cases.

Referring to Mr. Dana's comments on the writer's reasons for using attached web members because of the facility for repairing beams damaged by fire, it should be pointed out that exposure of steel members results from the spalling of the concrete, rather than from complete dehydration of the cement. The Baltimore fire, and many fire tests have demonstrated that the steel itself is often exposed without suffering serious damage; it is to be presumed that the fire is usually about exhausted, in such cases, before the concrete finally comes off. But, for fire-resisting structures, the writer is opposed to steel of very high elastic limit, because of the very danger pointed out by Mr. Dana. If every beam that has its main rods exposed has to be torn out and rebuilt, reinforced concrete will not long be in favor with underwriters, for the salvage will hardly be greater than in the case of a timber structure. While it is a slight digression, it might be suggested that if every fire test of a proposed type of fire-resisting construction had been carried far enough to determine correctly the salvage after the fire, a good many erroneous conclusions on the subject of fire-proof buildings would have been avoided.

Mr. Turner seems not to have comprehended the real intent of the paper at all; he has set up a straw man and knocked him over, but it is not clear that this has any very direct bearing on the questions discussed. However, an attempt will be made to answer Mr. Turner's objections.

There is decided room for differences of opinion as to whether the type of construction shown by Mr. Turner in Figs. 8 and 9 will be more economical than an equivalent one, using ribs or beams, and a thinner slab. Ingenuity in designing the centering for the ribs might upset Mr. Turner's estimates of cost, without the slightest trouble.

The writer is quite well aware, however, of the extra cost of centering for ribs, and, for that reason, has often preferred and used

Captain Sewell.

a plain slab rather than a ribbed construction, even though the slab was quite heavy. But he still maintains that for extensive floors, and heavy loads, there is economy in a ribbed construction; moreover, the conditions of the problem do fix the spacing of ribs within rather narrow limits, as a rule. However, it was not the purpose to treat this question by the laws of maxima and minima, for the reason that the writer has not been able to find any principles governing it in such a way that the skill of a good foreman or designer, expended on the centering, might not be the vital factor in deciding the question, after all. But, assuming the spacing of beams and girders to be fixed, there are such questions as the most economical design of slab and the most economical design of beam, and it was these questions that the writer undertook to discuss—that is, the economical sections of slabs and beams, the bending moments in the two cases being known. In the structure, as a whole, there may be broader questions of economy than in its individual members; but there is an economical design for each of them, and it should not be neglected, for it has a very vital bearing on the whole of the broader question.

Mr. Turner seems also to have missed the essence of the discussion of minimum cost for individual members. If concrete is very cheap, and steel very dear, it may easily happen that the cheapest beam, to carry a given load, may be one in which the concrete is not stressed to anything like its safe limit. On the other hand, it might be possible for concrete to be so expensive, and steel so cheap, that the least expensive beam would be one in which the steel was stressed very lightly, and the concrete to the full limit. Under such conditions, the paper might have to be revised; but, if such conditions existed, it would probably be cheaper to use structural steel; therefore no attempt was made to discuss the question under such assumptions. It is somewhat surprising, however, that this point has not been raised in the discussion of the paper.

As far as the utility of the writer's economic theory is concerned, he has found it useful in his own work, and detailed estimates based on actual designs have proven it correct within its own limits. To make its utility perfectly clear, one might suppose a piece of construction work in which the excavation for footings, etc., yields an ideal material for concrete; there may be nearby a cement plant, from which cement can be obtained at mill prices. Probably the concrete, apart from centering, in such cases, would cost not more than $2.25 per cu. yd. Suppose the cost of steel delivered at the site is very high, say, 5 cents per lb. It does not require a mathematical discussion to show that under such circumstances, economy demands deep beams and slabs, and light rein-

forcement; but there is an economical limit, and the writer's theory enables the engineer to determine it at once. It would seem that this is worth while, at any rate. Captain Sewell.

The writer is unable to see the peculiar value of his economical theory to any commercial interest, but as he has never been engaged in any commercial enterprise, there may be possibilities in it which his lack of experience prevents him from seeing.

Mr. Turner lays great stress upon the tests of certain floor slabs, presumably of his own design. Their behavior cannot be explained by any rational formula based on flexure, for the simple reason that they were never subjected to such a bending moment as Mr. Turner figures as due to the load. The secret of their great carrying power is the absolutely unyielding abutments afforded on all sides by the remainder of the floor construction. The writer has had exactly parallel cases in his own work, and has seen them in the work of others. If Mr. Turner had selected an outer panel, next to the wall, for his test, he might have obtained a very different result; or, if he had cut his test panel loose from the neighboring panels—or, if he had tested an entire floor to the extent illustrated in his photographs—it is quite certain that the results would not have been inexplicable, even by the theory of flexure. As a matter of fact, in the panel which was about 16 ft. square, the shearing stress around the outer edges appears to have been not more than 60 lb. per sq. in.—not a dangerous value for really good concrete. The panel, acting as a flat dome, was able to carry the load with stresses probably not exceeding 2 000 lb. per sq. in. The reinforcement below, and the load above, prevented local deformation or buckling. It is very doubtful, however, whether two or three times the load of 900 lb. per sq. ft. would have been required to produce collapse; in fact, the load, as it was, was rather dangerously close to the limit, and, also, under such conditions, collapse is almost sure to be sudden and without appreciable deflection or warning of any kind. Mr. Turner would have added much to the knowledge on the subject if he had carried the test to destruction.

The writer's opinions on the subject are derived from the study of one or two actual failures, where conditions were not entirely unlike those shown in Mr. Turner's photograph; but he will be very glad to revise them, if Mr. Turner, by a test or tests carried to destruction, can prove them wrong.

The writer's explanation of the carrying power of the slab is not materially different from that indicated by Mr. Turner himself, in Fig. 11. Both require unyielding abutments; the slightest movement in these would develop the bending stresses at once, and even Mr. Turner will hardly claim that his slabs would have resisted, as beams, the bending moments due to his loads. The

Captain Sewell.

writer has used slabs reinforced both ways by plain round rods, and supported all around by beams, as in Mr. Turner's tests; and, where he felt sure of unyielding abutments, he has reduced the bending moments for which the slab was designed, to an extent that he would not recommend for ordinary practice. As Mr. Turner does not disclose his own methods, he may feel the same way about them; but, any one familar with the carrying power of a flat Guastavino dome need feel no surprise at the carrying power of Mr. Turner's slabs, whatever may be the true explanation. It might also be suggested that steel beams tightly framed in between unyielding abutments, would carry loads as much in excess of their capacities, as based on the theory of flexure, as did Mr. Turner's floor slabs. The writer has often thought that, in insisting upon statically determinate conditions, American designers of steel structures have sacrificed a great deal of reserve strength, especially against local overloads, as well as a very desirable rigidity and a perfectly justifiable economy.

To the writer's mind, however, Mr. Turner's slabs were in a dangerous state of unstable equilibrium under the test loads; a very little fire, or a sudden shock, applied in addition to the loads, would probably have caused collapse of a very sudden and destructive nature. While the writer knows but little of the floor in the building of the Farwell, Ozmun and Kirk Company, to which Mr. Turner refers, he thinks that, in the absence of tests to destruction in both cases, final conclusions and invidious comparisons are not justified.

It is noted that Mr. Turner does not state the working loads for which the floors described by him were designed, nor does he reveal the methods used—all of which would have been much appreciated.

The basis of the writer's belief in attached web members is the very satisfactory practical results shown in their use. Professor Talbot's tests, referred to by Mr. Turner, were hardly fair, because the web members were not long enough, and they were not spaced according to the variation in stresses. The writer used the Warren truss as the analogy, because tests show that the web stresses in reinforced concrete, or any other solid, beams, are inclined at an angle of about 45°, and, as long as the beam is solid, they cannot be made to take any other direction; vertical web members can take up the vertical components of the tensile web stresses, but they cannot change the direction of the resultant stress. As for Mr. Turner's suggestions of an inverted bowstring or a Bollman truss, he would find some difficulty in anchoring his tensile members at the ends. In his application of the writer's economic theory to a plate girder made of two kinds of steel, he deduces results which would

be correct if, in steel, it were cheaper to let the unit web stresses vary; but, as a matter of fact, the conditions are entirely different, and the unit stress in the web would be the constant—not the thickness of the web. This application to an impossible—or at least highly improbable—plate girder, is merely another straw man demolished. Captain Sewell.

The suggestion that the lower part of a beam might be made of clinker concrete was not made with a view of reducing the first cost, but of improving the fire-resisting qualities.

Mr. Wason has made an extensive comparison of different formulas, most of which are convertible, one into the other, by changes in constants, and in the form of the stress-strain curve. The writer has no doubt that the percentages of steel worked out by himself are well within safe limits; he intended that they should be. If economy demanded it, he would not hesitate to increase them by at least 10 per cent. This, in the particular case assumed, would probably have given values more satisfactory to Mr. Wason.

However, Mr. Wason's mathematical work is not quite consistent. In his use of Professor Talbot's formulas, he assumes the value of A, overlooking the fact that there is a certain value of A which necessarily follows from his previous assumptions, and which Professor Talbot's formulas afford the means of determining. If Mr. Wason will take the writer's Equations 1 to 5, revise them to suit the assumption of a parabolic stress-strain curve with its vertex on the extreme fiber in compression, and assume $\frac{E_c}{E_s} = \frac{1}{20}$, he will get a set of formulas which are absolutely convertible into Professor Talbot's when developed under the assumptions made by Mr. Wason. He will find, also, that, with such a stress-strain curve, a value of $\frac{1}{10}$ for n in Professor Talbot's formulas is the same assumption as a value of $\frac{1}{20}$ for $\frac{E_c}{E_s}$ in the writer's formulas, revised for the parabolic curve.

Mr. Wason's own formula is based upon impossible assumptions. If the neutral axis is at the middle, the upper half—not the upper third—is in compression. Solving for working stresses, the right line is undoubtedly the correct form for the stress-strain curve. Under these circumstances, the maximum fiber stress in beams designed by Mr. Wason's formula is 667, instead of 500, lb. per sq. in. Moreover, in Mr. Wason's method, there is nothing to indicate that if he had used 100 000 instead of 50 000 for f, he would have obtained a different value of A; in which case, his formula would make the resisting moment of his beam 1 500 000, instead of 750 000 in-lb. Mr. Wason would not consider the beam

Captain Sewell.

good for that, but if his formula were written into a building law, some one else might. As a matter of fact, in his formulas, Mr. Wason does not take account of the elastic properties of the materials at all.

At the bottom of page 323, Mr. Wason makes an interesting calculation, which is a good argument for using a curved form of stress-strain curve, and solving for stresses near the ultimate, since the right-line-working-stress method puts the concrete at a manifestly unfair disadvantage. In Mr. Wason's computation of the compressive force of concrete, under the parabolic assumption, however, the coefficient, $\frac{5}{8}$, should be $\frac{2}{3}$. The coefficient, $\frac{5}{8}$, belongs, however, to a curve much closer to the truth than a parabola, and the lever arm of 0.8125 d is quite safe. There is no doubt that the beam assumed by Mr. Wason would easily carry 2 sq. in. of reinforcement, and that it would not then collapse under less than 1 000 000 in-lb., if properly designed and built.

It seems to the writer that compression in the steel, due to shrinkage of the concrete, is merely the tensile strength of the concrete in another form. It is an additional argument for attached web members.

The Watertown tests indicate that, with increasing age, the ultimate strength of the concrete increases more rapidly than the modulus of elasticity. This increases somewhat the factor of safety, but does not in any way affect the economy of the design.

The tests with loose diagonal stirrups, cited by Mr. Wason, seem to prove very conclusively the existence of the diagonal web stresses, and the necessity for rigid attachment of the web members, resisting them, to the main bars. Mr. Wason is entirely mistaken as to the difficulty of assembling and handling such reinforcement as was used at the War College. The web members were so rigid that the entire reinforcement was easily handled as a whole; the cost, in place, was about 2.4 cents per lb. The concrete was made of small gravel, and poured in, very wet. The writer does not believe that any one of the systems of web reinforcement described by Mr. Wason is as cheap or as easily handled and embedded as that used at the War College.

The equation deduced by the writer for the cost of the variable elements of a reinforced concrete beam is a hyperbola, which, when plotted, would almost coincide with the curve of total cost in Fig. 24, of Mr. Goodrich's discussion.

In answer to Mr. Goodrich's comments, on page 344, relative to the quantities, s, b, and c, this really raises the question of maximum economy in the spacing of ribs. With a given system of centering, this problem is capable of solution, but it would have

to be solved separately for each case. Practical considerations will generally restrict the number of bays into which a given space can be divided, but a mathematical discussion of minimum cost might be useful in each case, as indicating which of the available numbers of bays is most economical. Captain Sewell.

It seems to the writer that Mr. Goodrich's brick beams had a type of connection between web members and main bars which, when deflection had taken up the slack, became quite rigid, because of the friction. If he had set the U's the other way, the experiment would have been more conclusive. As it was, it appears to be quite as much in favor of his contentions as against them. If the stirrups of the beams shown in Plate XXVI were wrapped closely around the main bars, the same would be true of them.

Mr. Goodrich, as well as Mr. Turner, expresses some doubt as to the correctness of the ordinary assumptions made in designing reinforced concrete structures. It will not be denied that, for isolated beams, the theory of flexure explains the results of tests fairly well. Slabs reinforced in two directions and supported on all sides may give results a little higher than would be indicated by the theory of flexure as ordinarily applied, and this may be due, in some measure, to the counterbalancing effects of compressive stresses acting at right angles to each other, on the same material. But it is not clear how the two sets of reinforcing bars could relieve each other of tensile deformation in the same way, so that an isolated slab built and supported as described, would probably not carry much more than the breaking loads found by the theory of flexure, rationally applied. Any increase in strength could be allowed for by a judicious reduction of applied moments in designing.

When a slab or beam is rigidly built in as part of an extended structure, however, there is a very great increase of strength under tests applied to only one or two units of the floor system at a time, especially when they are interior units. Exactly the same results have been attainable with steel structures, any time during the last twenty years, but no one has thought it advisable to do so, and count upon them. If not advisable for a tough and ductile material like steel, it is much less advisable for a brittle material like concrete. If dome action is to be counted upon, then build domes, self-contained, each within its own tension ring—not slabs and beams, dependent upon unloaded and undamaged neighbors for capacity to carry, as domes, loads that applied to them, as beams, would instantaneously and completely destroy them. Of continuous girder action, however, the writer would take fairly full advantage; and he is convinced that it would have been advisable to do the same thing in steel structures for these many years.

Captain Sewell.

In answer to Mr. Goodrich's claim that a sufficiency of tests is available to settle disputed points, it might be suggested that the manifest differences of opinion among those conversant with the subject is pretty good proof to the contrary. Beams exactly alike in all particulars except as to the type of web reinforcement and its method of attachment to the main bars, should be tested accurately, in a testing machine, with numerous points of contact, well distributed throughout the span, to prove whether the writer's theory of web reinforcement is or is not correct. Practical tests are not sufficiently accurate to be conclusive and to remove the question from the domain of opinion to that of established fact.

The writer desires to express his appreciation of Mr. Goodrich's very discerning, fair, and frank discussion.

Mr. Thacher's objections to the writer's proposed multiplied loads would apply to bridges and other structures in which the dead load is the principal load. A fair average case in a building would be one in which the dead and live loads are about equal. The writer's proposition, in such a case, involves a safety factor of $2\frac{1}{2}$, based on conditions at a stage materially short of collapse. It is very doubtful whether the floor systems of most steel-frame buildings have any such factor; the tile arches in common use would collapse before the stress in the steel beams reached the elastic limit—and this would fix the factor at very little more than 2.

If reinforced concrete is made a little safer than the type of structures it is trying to displace, it is sufficient. Where the dead load becomes very great, however, there should be adopted a sort of sliding scale, whereby the factor of safety, based on total loads, would never be less than $2\frac{1}{2}$. For structures subject to shock and vibration, it should be increased.

The writer naturally dissents from Mr. Thacher's views as to the adequacy of the concrete for binding main and web members together. However, if there is any economy in omitting the surplus concrete, in a completely reinforced beam, there is no objection to doing so, provided adequate fire resistance is not sacrificed.

In reply to Mr. Forchhammer, attention is called to the fact that the stress-strain curve used in deducing Equations 1 to 5 is correct only on the assumption of a certain maximum stress in the concrete; to allow this stress to vary as Mr. Forchhammer does, and still use the equations with the constants deduced by the writer, is manifestly incorrect. Mr. Forchhammer, in arriving at a beam of minimum cost, goes through all the tentative calculations, which the writer tried to avoid, and deduces no rule of general application.

The writer cannot see that his use of the words, "maximum allowable percentage," is misleading when they are read with the

context, wherein it is plainly stated that all percentages are determined so as to cause the beam to fail by failure of the steel. Captain Sewell.

"Theoretical economy based on relative costs," is attained when the cost of the steel and the cost of the concrete above it are equal. If this gives so great a percentage of steel that the concrete will fail before the stress in the steel reaches the elastic limit, the beam is not able to resist the moment used in determining the area of the steel. A deeper or wider beam of greater cost is demanded. Therefore, the beam of least cost, determined solely by consideration of the relative cost of the two materials is not adequate, from a structural point of view, and cannot be used. The maximum attainable economy, of course, results from using the "maximum allowable percentage" of steel. When more than this is used, the strength of the beam is increased, but not in proportion to the increased area of steel nor to the increased cost. It may not be quite accurate to say that the increase in the steel is all "wasted," but it is certainly not economically used. There may be cases where dead weight must be avoided at any cost—or where minimum thicknesses must prevail—in which the design must be based on the use of very large percentages of steel; but ordinarily this is not necessary. Where the strength of the concrete determines the failure of the beam or slab, the failure is apt to be sudden, and to come without much warning. This is a very undesirable kind of failure, and should be avoided, if possible. As already pointed out, if concrete is so expensive that it is cheaper to design on the basis of the strength of the concrete, it is probably still cheaper to use structural steel and some form of fire-proof floor arch, such as hollow tiles.

In reply to Mr. French, there is no objection to designing by the straight-line formulas, for working stresses, provided values are assumed for $\frac{E_c}{E_s}$ and F, which will be as near the truth as possible, and at the same time, will permit of the use of reinforced concrete with a safety factor only a little greater than that existing in structural steelwork designed to carry the same loads. Building departments will not ordinarily approve designs frankly made with such values of $\frac{E_c}{E_s}$ and F—especially the latter—yet they will pass lighter designs made in accordance with a purely empirical formula in which a fictitious working stress of 500 lb. or less is one of the factors, and an utterly impossible value for the area of compression is another. The writer frankly admits that his chief reason for recommending the method of multiplied loads and ultimate stresses is the hope that it may the more easily lead to designing by honest formulas, with reasonable factors of safety. The writer often designs for working stresses, but, fortunately, does not have to reckon with any building department.

Captain Sewell.

The writer is much indebted to Professor Church for working out the more general solution of his problem in maxima and minima, thereby avoiding the slight inaccuracy of assuming a fixed value for the coefficient, h. That the writer himself evaded this issue should, at least, relieve him of the suspicion of being a mathematical gymnast, which seems to have been aroused in the minds of some by the very modest mathematical exercises contained in the paper. It should be pointed out, however, that the more rigid method followed by Professor Church, when applied to the practical case in which there is a minimum, states the conditions of minimum cost in terms of relative stresses in the steel and concrete. To determine the actual area of steel in the design of minimum cost would then involve the solution of Equations 1 to 5. The writer's method does not give, with rigid accuracy, the theoretical minimum; but it comes close enough for all practical purposes, and gives the result in more convenient form for use.

The question of combined thrust and moment raised by Mr. Leffler is one of extreme interest. This was not discussed in the paper because the writer does not believe in relying upon arch action in floor systems as ordinarily designed. Therefore, he confined himself to a consideration of stresses due to bending only. The simplified formula recommended by the writer is just as applicable to working stresses as to ultimate stresses, but, as it is concerned with the steel alone, it would not serve for the combined thrust and moment problem, without the use of special values of $\frac{a}{d}$.

Replying to Mr. Hill, the writer would not recommend the duplication of his own mathematical work in any case; the general rule that minimum cost will result when the cost of the steel is as nearly as possible equal to that of the concrete above it, is the only practical result of the application of the calculus, and it is merely one point that should be kept in mind, along with the cost of centering, variations in the prices of steel and cement, caprices of the labor unions, etc.

In reference to the dehydration of cement, mentioned by Mr. Hill, if attached web members are not used, concrete which is at all dehydrated would certainly not be reliable for transmitting stresses into the steel. Even if attached web members are used, the concrete surrounding the main bars will take some stress; although this action is not counted upon, it cannot be entirely avoided. Dehydrated concrete would be very apt to crack and fall off, under such circumstances; in any case, unless it is removed and good material substituted, the damage is not wholly repaired, and the structure is not as good a fire risk as it was before.

Mr. Hill's remarks in reference to methods of making tests, etc., are of the utmost importance, and should receive most careful consideration.

Captain Sewell.

Mr. Shearwood also raises some extremely important points, which should be settled by careful experimental investigation. The point about the effect of repeated loads is one of great interest; the writer believes that experiments along this line will demonstrate the value of attached web members, but frankly admits that, thus far, this is only an opinion.

In reply to Professor Merriman, the writer cannot see any logical difference between designing for conservative values of ultimate stresses, with a factor of safety, and designing for working stresses, which are determined by dividing ultimate stresses by an assumed factor of safety. What is illogical in one method, is equally so in the other.

The writer has not had time to analyze Professor Merriman's mathematical work, but as it is apparently based on the full utilization of the concrete, in all cases, instead of the steel, it necessarily leads to results different from those deduced in the paper. Professor Merriman, however, has not proven that his method gives more economical beams than the writer's method. As for uncertainties, there is nothing more uncertain than the correct value of $\frac{E_c}{E_s}$. The writer thinks that a sufficient number of well-conducted experiments would put a formula, such as that which he has proposed, into such shape as to be much more reliable than any in which $\frac{E_c}{E_s}$ is a factor.

The form of formula recommended in the paper, has no necessary connection with Equations 1 to 5. The writer recommends abandoning the latter, and all others like them. Equations 1 to 5 were used to determine tentative values of the constants entering the formula, $M = h\ t_s\ a\ d$.

If a sufficient number of well-considered experiments could be made, the constants should be determined from them.

In reply to those who fear that the writer's proposed formula would lead to dangerous results, he would say that it is clearly stated in the paper that limiting values of $\frac{a}{d}$ should be properly determined, and always kept in mind. If this is done, dangerous designs cannot possibly follow.

The writer does not expect the impossible from web reinforcement, but he does expect that it will prevent sudden collapse, in any event, and will prevent total collapse until after the stress in the steel has passed the elastic limit; just how far, is still a matter for determination. But, in any case, it will put reinforced concrete, in the writer's opinion, on the same basis as steel beams and terra-cotta arches, for safety: the arches would almost surely fall out, and the beams be distorted so that they would have

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to be taken out and strengthened or renewed, shortly after the stresses in them passed the elastic limit. With complete web reinforcement, the reinforced concrete structure would be, at least, no worse off.

The writer regrets that his present duties prevent him from replying in more detail to the various interesting criticisms of his paper, but thinks, on the whole, that the essence of all that could be said, is really contained in the paper itself and in this discussion. Probably it is just as well to close the discussion here; the writer hopes that experiments will soon be made in an authoritative way, which will make a future discussion, such as this, quite impossible.

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TRANSACTIONS.

Paper No. 1024.

NEW FACTS ABOUT EYE-BARS.*

By Theodore Cooper, M. Am. Soc. C. E.

With Discussion by Messrs. Henry B. Seaman, Mansfield Merriman, Albert J. Himes, A. W. Carpenter, John Thomson, Mace Moulton, John D. Van Buren, J. W. Schaub, H. de B. Parsons and Theodore Cooper.

When, in the course of professional practice, new facts are discovered which either broaden or contradict previously accepted beliefs, it is a professional duty to present the results and deductions, after careful examination, for the common benefit of our fellow workers.

"Prove all things; hold fast that which is good."

In the execution of the superstructure of the Quebec Bridge, with its 1 800-ft. channel span, the great magnitude of the members, the high working strains, and other features of the work, have demanded careful study of many points, which, in ordinary bridges, could be and have been overlooked or neglected as of small importance.

This paper will be confined to the new facts developed in regard to eye-bars.

It was a surprise and a cause of much anxiety to the writer to discover how defective was our knowledge of the eye-bar.

As a general rule, we have failed to recognize that the real elongation of an eye-bar is from out to out of pin-holes, and not

* Presented at the meeting of March 21st, 1906.

from center to center of pins. We have carefully determined its elongation and elastic limit over a certain length of the parallel bar, and then accepted this determination as equally true when applied to the whole bar. We have assumed that a set of bars carefully bored to an exact length would all pull to an equal strain, as long as the elastic limit measured on the body of the bar was not exceeded. All these beliefs and assumptions are incorrect.

In ordinary bridges this has not been a matter of much importance, owing to the low unit strains and the small change in the deformations.

Description of the Investigation.

In August, 1904, the manufacture of the eye-bars for the anchor arms of the Quebec Bridge was well under way when the question arose as to what clearance should be allowed between the pins and pin-holes. The eye-bars forming the tension members were 15 in. in width, from $1\frac{1}{4}$ to $2\frac{1}{16}$ in. in thickness, and of lengths from 50 to 58 ft. The pins, except in a few special cases, were 12 in. in diameter and from 8 to 10 ft. in length. The maximum joint has 58 bars on one pin.

After discussion, it was decided that the clearance necessary for the purposes of erection should not be less than $\frac{1}{32}$ nor more than $\frac{1}{16}$ in., the latter being about the proportion given to ordinary bridge pins.

As the maximum working strains are higher than in usual practice, being about 21 000 lb. per sq. in. in tension, the intensity and distribution of the local pressures from the pin to the eye of the bar became important.

The writer, after a little consideration of the problem, realized that, while the local pressure must be very great, and the pin-holes must deform elliptically, at least elastically and probably permanently under the proposed working strains, the solution could only be obtained by experiment. He then devised a method of measuring the elongation of the bar from out to out of pins, while the bar was strained to varying amounts in the testing machine.

Four bars with different clearances were prepared and tested to 12 000, 16 000, 20 000, 24 000 and 28 000 lb. per sq. in.

While the result of these tests (Nos. 646, 647, 648 and 649 of Table 2) was not fully satisfactory, owing to the crudeness of the

hurriedly made appliance, and the difficulty of reading the fine measurements in the limited space, they showed that the bars, from out to out of pins, began to elongate permanently at 12 000 lb., and that the elongation increased with each increase of strain. The amount of the pin clearance did not modify the results especially.

Was this deformation, even at low strains, a peculiarity of this "make" of bars, or had it been observed in other tests? Looking up old records, the writer found in his abstract of tests made at the St. Louis Bridge in 1872, that of 58 eye-bars put to the proof test of 18 000 lb. per sq. in., 5 showed a permanent elongation of the pin-holes of $\frac{1}{16}$ in., 52 of them $\frac{1}{32}$ in., and 1 of them $\frac{1}{64}$ in. These were iron bars.

In the Watertown "Tests of Metals" for 1883, there were found tests on 6 steel eye-bars, where the permanent elongation between pin centers at different strains is noted. They are abstracted in Table 1.

TABLE 1.—BARS, $6\frac{1}{2}$ BY 1 IN.; PINS, 5 IN.; EXCESS, 40 PER CENT. AT SIDES; END SECTION, 86 PER CENT.

No. of Bar.	STRETCH OF BAR, OUT TO OUT OF PIN-HOLES, IN INCHES.						Ultimate Strength.
	10 000	20 000	25 000	30 000	35 000	40 000	
4582	0.015	0.020	.....	.....	0.090	2.72	67 800
4583	0.010	0.020	0.025	0.040	0.060	2.63	64 000
4584	0.015	0.020	0.025	0.040	0.050	....	65 000
4585	0.020	0.025	0.030	0.045	0.070	....	65 850
4586	0.020	0.030	0.030	0.045	0.175	....	64 400
4587	0.010	0.015	0.025	0.030	0.060	....	68 290

It became evident, therefore, that the stretch of the eyes of eye-bars was not peculiar to the present "make" of bars, but had always occurred.

As it was important to push the construction of the work, it was decided to give the 12-in. pins $\frac{3}{64}$ in. clearance, and, for the anchor arms then under construction, to add an extra allowance of $\frac{1}{32}$ in. to the elastic elongation of each eye-bar for camber determinations; and should the further and fuller tests, then determined upon, show this to be too much or too little, the correction could be made in the cantilever arms.

TABLE 3.

No. of bar.	Thickness, in inches.	Head.	Excess, percentage.	Pin clearance, in inches.	Stretch of Pin-Holes, in Inches, at Varying Strains per Square Inch.						Stretch, Out to Out of Pin-Holes, by Tape.	Ultimate strength of bar.	Heat number.	Specimen Tests of Heat.		Remarks.
					16 000	18 000	20 000	24 000	26 000	35 200	24 000			Tensile strength.	Elongation, percentage.	
760	1¼	A	62	0.049	0.017	0.017	0.018	0.060	0.090	0.614	7/64	.....	14 215			
		B	66	0.054	0.019	0.020	0.024	0.048	0.072	0.666						
761	1¼	A	56	0.074	0.009	0.013	0.030	0.085	0.136	0.726	9/64	.....	14 215	61 180	24.5	
		B	59	0.053	0.000	0.005	0.015	0.037	0.066	0.649				64 020	26.5	Cut from same bar.
762	1¼	A	52	0.065	0.019	0.032	0.039	0.090	0.138	0.942	11/64	.....	14 215	65 640	24	
		B	59	0.050	0.009	0.012	0.016	0.039	0.055	0.720						
763	1¼	A	55	0.054	0.000	0.008	0.016	0.033	0.056	0.821	3/32	.....	14 215			
		B	51	0.052	0.009	0.015	0.022	0.062	0.105	0.740						
										Ultimate.					
798	2	A	47	0.017	.....	.....	0.017	0.035	.....	3.0	7/64	56 430	12 937	60 150	26.5	
		B	45	0.054	.....	.....	0.000	0.020	.....	3.8				61 860	29	" " " "
799	2	A	46	0.044	.....	.....	0.011	0.042	.....	3.2	7/64	56 740	12 937	65 180	26.5	
		B	41	0.047	.....	.....	0.016	0.029	.....	2.8						
800	1 7/16	A	47	0.063	.....	.....	0.028	0.065	.....	3.7	1/8	57 780	9 919	66 320	24	
		B	50	0.052	.....	.....	0.038	0.057	.....	3.8				67 220	23.5	" " " "
801	1 7/16	A	49	0.050	.....	.....	0.032	0.056	.....	3.6	7/64	57 080	9 919	69 220	23	
		B	47	0.049	.....	.....	0.024	0.055	.....	3.7						
802	1 9/16	A	43	0.049	.....	.....	0.032	0.061	.....	4.4	3/16	53 800	16 099	66 580	26	
		B	47	0.053	.....	.....	0.027	0.069	.....	3.9				66 670	27	14-in. pins.
803	1 9/16	A	46	0.050	.....	.....	0.022	0.050	.....	4.3	1/8	54 300	16 099	68 050	23	Cut from same bar.
		B	47	0.051	.....	.....	0.023	0.049	.....	4.8						
804	2	A	44	0.056	.....	.....	0.029	0.058	.....	3.9	7/64	59 500	16 093	64 900	25	
		B	44	0.046	.....	.....	0.019	0.034	.....	4.3				65 230	25.5	*A* pins 12 in.—*B* pins 14 in.
807	2	A	46	0.053	.....	.....	0.018	0.036	.....	3.0	7/64	58 800	16 093	65 500	26	Cut from same bar.
		B	47	0.033	.....	.....	0.010	0.079	.....	4.0						
808	1 3/4	A	41	0.056	.....	.....	0.008	0.043	.....	3.9	5/64	58 240	12 891	63 180	29	
		B	42	0.062	.....	.....	0.012	0.036	.....	2.8				64 260	27	Cut from same bar.
809	1 3/4	A	44	0.054	.....	.....	0.008	0.030	.....	3.2	5/64	57 670	12 891	67 170	24	
		B	48	0.049	.....	.....	0.008	0.034	.....	3.1						
822	1 5/16	A	48	0.052	.....	.....	0.010	0.019	.....	3.5	3/64	60 940	8 690	60 370	28	
		B	48	0.057	.....	.....	0.012	0.025	.....	2.9				62 280	28	" " " "
823	1 5/16	A	51	0.044	.....	.....	0.017	0.032	.....	3.5	5/64	60 440	8 690	64 500	25	
		B	47	0.070	.....	.....	0.021	0.034	.....	4.8						
759	2 1/8	A	58.6	0.046	0.000	0.007	0.017	0.012	0.025	.....	7/64	.....	.....	60 000	25	Riveted links, made of plates.
		B	Gross.	0.026	0.016	0.020	0.040	0.050	0.064	.....						
758	2 1/8	A	58.6	0.060	0.000	0.007	0.010	0.022	0.027	.....	3/32	.....	.....	63 400	24.5	
		B	Gross.	0.058	0.000	0.013	0.017	0.028	0.037	.....						

TABLE 2.

No. of bar.	Thickness, in inches.	Head.	Excess, percentage.	Pin clearance, in inches.	Stretch of Pin-Holes, in Inches, at Varying Strains per Square Inch.						Stretch, Out to Out of Pin-Holes, by Tape.	Ultimate strength of bar.	Heat number.	Specimen Tests of Heat.		Remarks.
					12 000	16 000	21 000	24 000	28 000	Rupture.	24 000			Tensile strength.	Elongation, percentage.	
646	1¼	A	44	0.033	0.037	0.052	0.055	0.113	0.138	3.6	.....	60 900	14 069	63 970	27	These were the first bars tested. The results given are only approximate, as the gauge appears to have slipped, especially in 648 and 649. *Untrustworthy.
		B	44	0.035						3.6				65 180	27.5	
647	1¼	A	54	0.033	0.010	0.011	0.020	0.021	0.020	3.2	.....	61 220	14 069	66 800	30	
		B	44	0.031						3.3						
648	2	A	48	0.072	0.070*	0.062*	0.089*	0.103*	0.088	3.4	.....	56 200	15 281	60 720	26	
		B	57	0.069					0.038*	3.1				64 730	28	
649	2	A	39	0.057	0.012*	0.060*	.....	.....	0.114*	5.1	.....	57 000	15 281	65 320	25	
		B	50	0.060						3.4						
705	1¼	A	52	0.073	0.014	0.017	0.033	0.081	.....	1.7	⅜	51 230*	16 672	60 300	31	Broke at flaw in head B. 704 cut from same bar, broke at 57 190 and pin holes elongated 3.5 and 3 in.
		B	49	0.073	0.010	0.029	0.072	0.205	.....	2.4*				60 300	32	
														62 560	30	
706	1¹⁵/₁₆	A	48	0.064	0.009	0.044	0.112	0.266	.....	3.6	³⁷/₆₄	57 730	7 666	61 380	29.5	Cut from same bar.
		B	48	0.073	0.000	0.035	0.084	0.207	.....	3.5				61 880	27	
707	1¹⁵/₁₆	A	42	0.082	0.006	0.013	0.045	0.148	.....	3.1	³³/₆₄	55 160	7 666	66 000	26	
		B	46	0.075	0.010	0.028	0.055	0.135	.....	3.0						
708	1¹³/₁₆	A	40	0.050	0.004	0.006	0.012	0.032	.....	2.5	½	59 450	13 250	65 980	24	" " "
		B	39	0.048	0.000	0.017	0.044	0.059	.....	3.7				66 220	25	
709	1¹³/₁₆	A	31	0.084	0.000	0.016	0.043	0.106	.....	3.0	½	58 340	13 250	66 760	26	
		B	43	0.075	0.016	0.020	0.035	0.105	.....	3.3						
710	1¹⁵/₁₆	A	53	0.072	0.005	0.005	0.009	0.015	.....	1.7	³²/₆₄	60 230	14 069	63 970	27	
		B	44	0.075	0.003	0.012	0.025	0.042	.....	3.2				65 180	27.5	
711	1¹⁵/₁₆	A	49	0.051	0.000	0.000	0.000	0.031	.....	2.1	0	58 960	14 069	66 800	30	
		B	48	0.070	0.000	0.000	0.000	0.045	.....	2.6						
712	1¹⁵/₁₆	A	46	0.034	0.002	0.009	0.014	0.023	.....	2.5	³¹/₆₄	63 210	14 069			Cut from same bar as 716.
		B	40	0.037	0.000	0.007	0.012	0.027	.....	3.8						
713	1⅞	A	58	0.051	0.000	0.002	0.004	0.012	.....	2.0	1/32	65 220	4 853	64 380	23	Cut from same bar.
		B	54	0.054	0.000	0.000	0.005	0.006	.....	1.4				65 200	23	
714	1⅞	A	55	0.064	0.008	0.008	0.014	0.037	.....	2.1	³¹/₆₄	65 220	4 853	67 800	24.5	+ Broke at flaw in head B. Cut from same bar as 712.
		B	54	0.063	0.000	0.002	0.006	0.024	.....	2.3						
716	1¹⁵/₁₆	A	69	0.038	0.005	0.006	0.006	0.007	.....	1.1	1/64	57 800+	14 069			
		B	61	0.043	0.000	0.000	0.003	0.011	.....	.....						
717	1¹⁵/₁₆	A	49	0.048	.....	.....	0.010	0.028	.....	3.2	³¹/₆₄	64 360	14 074	61 780	29	Cut from same bar.
		B	42	0.060	.....	.....	0.008	0.029	.....	3.1				65 440	25.5	
718	1¹⁵/₁₆	A	59	0.064	.....	.....	0.006	0.019	.....	2.5	1/32	64 870	14 074	67 000	28.5	
		B	63	0.043	.....	.....	0.002	0.009	.....	2.4						

The importance of the permanent stretch of the eyes, as affecting the structure in other directions, was not overlooked, but the data so far obtained were too few to furnish any definite conclusions.

Preparations were made for fuller tests, and, to eliminate the difficulties of the first method of measuring the stretch, and also to get the action of each eye independently, the following method was adopted: Measure each bar from out to out of eyes, and calliper each eye longitudinally and transversely before putting it in the testing machine. Then, after straining the bar to 12 000, 16 000, 20 000, etc., lb. per sq. in., remove it from the machine and repeat the measurements.

In preparing the bars for test, it was determined to get from bars already made such a selection as would give a wide range in "heat numbers," "thicknesses," "proportions of the head," and "pin clearances." Some of the bars were specially bored to change the proportions of the head and the pin clearances, and two bars with visible flaws in the head were selected.

This selection covers Bars Nos. 705 to 718. The later bars are those which have since then been selected from time to time for the usual proof tests.

In Tables 2 and 3 all the important data of the tests so far made have been entered.

The records have been given as recorded. It will be noticed that the tape measurements from out to out of eyes, while they agree reasonably well with the sum of the elongations of the two eyes in most cases, differ in other cases. This may be partly due to errors of measurement and partly due to the measurements being taken on one side of the bar only; which, in the case of the bar being warped by the strain, would not give the exact length.

It should also be noted that in taking out and replacing the bar, if it did not get the exact position it first occupied on the pin, there would be an additional elongation of the hole before it got its proper bearings.

A number of the bars were additionally tested by trying to maintain a constant strain for several hours, to determine the effect of time. There was an increase of stretch, but it is believed to be at least partially due to the difficulties of holding a constant pressure on the machine for a long time.

To illustrate the method of the testing, one detailed test is here given:

February 8th, 1905. Test No. 711. Bar, 15 by $1\frac{9}{16}$ in. Heat number, 14 069.

Head A.	Head B.	
21.46 by 1.64 in.	21.60 by 1.62 in.	Elastic limit, 32 850 lb.
Excess, 49.4%.	Excess, 48.5%.	Ultimate strength, 58 960 lb.
Original area of bar, 23.56 in.		Fracture, 40% silky, 60% fine granular, half cupped.
Fractured area of bar, 13.66 in.		

Diameter of testing machine pin, 11.98 in.

TABLE 4.—Detailed Observations.

Strain per square inch, in pounds.	Diameters of Pin-Holes.		Out to out of pin-holes	10 ft. on body of bar.
	A.	*B.*		
0	*T* 12.032 *L* 12.031	*T* 12.050 *L* 12.050	15–9⅞	10
12 000	*T* 12.032 *L* 12.031	*T* 12.050 *L* 12.050	..	..
16 000	*T* 12.032 *L* 12.031	*T* 12.050 *L* 12.053	..	..
20 000	*T* 12.032 *L* 12.031	*T* 12.050 *L* 12.060	..	..
24 000	*T* 12.031 *L* 12.040	*T* 12.048 *L* 12.075	..	..
24 000 after first 2 hr	*T* 12.031 *L* 12.040	*T* 12.047 *L* 12.084	15–9$\frac{13}{32}$	..
24 000 after second 2 hr.	*T* 12.030 *L* 12.064	*T* 12.047 *L* 12.095	15–9⅝	10–0$\frac{1}{32}$
24 000 after third 2 hr.	*T* 12.030 *L* 12.064	*T* 12.047 *L* 12.095	..	..
After rupture	*T* 12.00 *L* 14.12	*T* 12.00 *L* 14.66	18–7$\frac{1}{2}$	12–28

After a number of tests had been made, from 12 000 to 24 000 lb., it was found that the important data could be obtained with less frequent removals, and thus save much time and labor. Therefore, with the exception of some special tests, the records afterward were taken only at 20 000 and 24 000 lb.

In testing two connecting links, Nos. 758 and 759, made of plates riveted together, four bars, Nos. 760-763, were used to make connection with the testing machine. The data for these members were extended over a wider range, but not carried to rupture, as the un-

supported eyes of the links began to buckle in advance of the pins at the higher strain.

On plotting the results, using the stretch of the holes after rupture as the upper limit, it was found that the stretch of the eyes at various strains per square inch followed a regular curve, differing for the different bars, but all having one general form, Fig. 1.

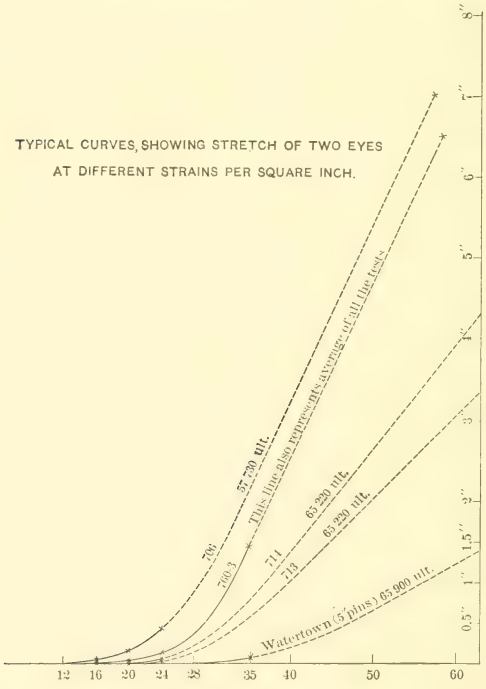


FIG. 1.

In order to save confusion by plotting each particular curve, the bars have been arranged in eight different classes, according to their stretch (covering both eyes). The average of each class is plotted in Fig. 2 on a larger scale than Fig. 1.

In addition to the previous observations, a number of the heads were scribed with fine lines, longitudinally and transversely, dividing the heads into spaces 2 in. square. After the rupture of the bars, these lines were traced and plotted, for comparison with the plots

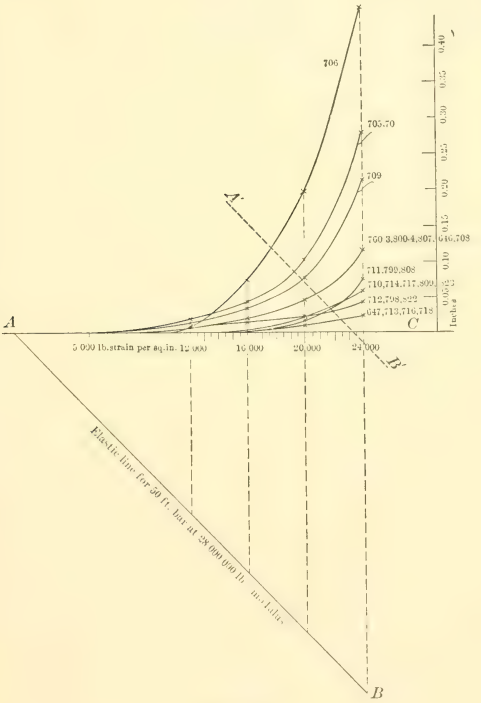


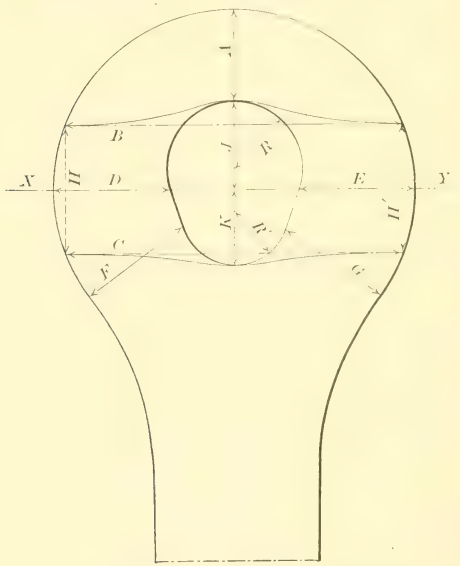
FIG. 2.

of the lines on the original bars, to determine the relative flow of the metal in different parts of the head.

It would be difficult to reproduce these tracings on a small scale. In Fig. 3 the principal and important changes from the original dimensions are shown, and the values are given in Table 5.

The line, $X\ Y$, is the transverse line through the center of the original pin-hole. The curved lines tangent to the elongated pin-hole are the forms taken by the straight lines tangent to the original hole before straining the bars. The upper part of the pin-hole is held to the form and diameter of the pin and has elongated more than the lower half. The diameter of the lower half has decreased by the

DEFORMATION OF HEADS OF EYE-BARS UNDER RUPTURE.



X-Y is line through center of original pin-hole

FIG. 3.

transverse closing in of the material under the pull. At the top of the pin the metal of the head is decreased in depth by the compression. The transverse dimensions across the eye and neck are reduced by the flow of the metal under tension. It is interesting to note that, with these proportioned heads, the distances, H and H', on the outsides of the heads opposite the pins, have elongated very little or not at all, which would indicate that the periphery of the

heads at these points had not been strained much more than the elastic strength of the metal, though the bar had been strained to rupture. It should be noted that in some of the 10 and 8-in. bars this distance on one side has been decreased, indicating a compressive distortion on one side. As, in two cases, it was as much as $\frac{1}{16}$ and $\frac{5}{64}$ in., it would not appear to be due to errors in measurement, the portion of the metal most severely taxed being that portion of the intrados of the eye lying between the horizontal and curved lines at the top of the pin.

The head, No. 708 *A*, pulled unequally in the neck, one side, *G*, decreasing $1\frac{1}{4}$ in., while the other side, *F*, only decreased $\frac{3}{8}$ in., showing softer metal at one side than the other.

The difference in the pulling of the heads of No. 710 may be partly due to the fact that No. 710 *A* was thicker, having an excess of 53%, while No. 710 *B* had only 44 per cent.

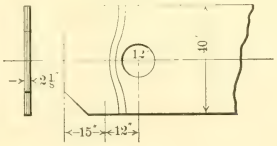


FIG. 4.

Similar observations were made upon some 8 and 10-in. bars, with like results.

The heads of the riveted links, not strained to rupture, showed a different action.

All the transverse lines below the center of the pin and those more than 12 in. above the center of the pin moved in a parallel direction, while those from the center of the pin to about 12 in. above took a curved form, as shown in Fig. 4.

This was due to the absence of any neck below the pin, and to the greater stiffness of the material above the pin to resist bending.

CONSIDERATION OF THE RESULTS.

As far as relates to the original purpose of the first tests, *viz.*, to determine the effect of the pin clearances, the tests give no definite answer. The different pin clearances vary from 0.031 to 0.084 in.

TABLE 5.

Bars.	Pins.	No. of Head.	Diameter of Head.	Deformation on Lines. (See Fig. 3.)											
				A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*H'*	*J*	*K*	*R'*
	Inches.		Inches.	In Sixty-fourths of an Inch.									Inches.	Inches.	Inches.
15 in.....	12	708*A*	33	−16	−48	−88	−40	−40	−24	−80	+0+	+0+	$+1\frac{5}{8}$	$+\frac{7}{8}$	5
		710*A*	$33\frac{7}{16}$	−16	−12	−32	− 8	−16	−16	−20	+0+	+0+	$+1\frac{1}{16}$	$+\frac{13}{16}$	$5\frac{1}{16}$
		710*B*	$33\frac{4}{8}$	−48	−64	−68	−48	−48	−64	−48	+0+	+0+	$+2\frac{1}{8}$	$+1\frac{1}{8}$	$4\frac{3}{8}$
		713*A*	35	−32	− 8	−20	− 0+	− 0+	− 0+	− 0+	+0+	+0+	$+1\frac{3}{8}$	$+\frac{5}{8}$	$5\frac{3}{8}$
		713*B*	$34\frac{8}{8}$	− 7	−20	−24	−12	−12	−10	− 0+	+4	+4	$+\frac{13}{16}$	$+\frac{11}{16}$	$5\frac{1}{16}$
		714*A*	34.9	−28	−32	−48	−24	−24	− 8	−16	0	0	$+1\frac{1}{16}$	$+\frac{13}{16}$	$5\frac{3}{16}$
		714*B*	34.75	−28	−16	−44	−16	−16	− 0	−16	0	0	$+1\frac{1}{8}$	+1	$5\frac{1}{16}$
10 in.....	$7\frac{31}{32}$	$1\frac{3}{8}A$	$22\frac{5}{16}$	−32	−19	−14	− 4	−44	− 0	−16	+3	−4	$+1\frac{13}{64}$	$+\frac{45}{64}$	$3\frac{11}{64}$
		$1\frac{3}{8}B$	$22\frac{29}{64}$	−36	−64	−56	−14	−24	−18	− 3	+3	+3	$+1\frac{15}{32}$	$+\frac{49}{64}$	$3\frac{17}{64}$
		$1\frac{1}{4}A$	$22\frac{13}{32}$	−28	−27	−45	−12	−23	−18	− 3	0	0	$+1\frac{17}{64}$	$+\frac{25}{32}$	$3\frac{25}{64}$
		$1\frac{1}{4}B$	$22\frac{1}{16}$	−28	−19	−14	−18	−19	− 9	− 3	+3	−5	$+1\frac{9}{64}$	$+\frac{25}{32}$	$3\frac{1}{64}$
8 in.....	$6\frac{5}{16}$	$1\frac{1}{8}A$	$18\frac{13}{32}$	−12	−18	−18	− 8	− 9	− 3	− 3	0	0	$+1\frac{1}{64}$	$+\frac{49}{64}$	$2\frac{51}{64}$
		$1\frac{1}{8}B$	$18\frac{9}{32}$	−14	− 8	−16	−10	−10	−11	− 3	−1	+2	$+1\frac{3}{64}$	$+\frac{51}{64}$	3
		$1\frac{3}{4}A$	$18\frac{5}{32}$	−16	−20	−20	−14	−10	− 5	− 5	+2	+2	$+1\frac{1}{32}$	$+\frac{1}{2}$	$2\frac{33}{32}$
		$1\frac{3}{4}B$	$18\frac{3}{8}$	−18	−34	−27	−12	− 8	−13	− 2	−1	+6	+1	$+\frac{5}{16}$	$2\frac{5}{16}$

− Decrease. + Increase.

While, no doubt, the pin clearance has some influence on the stretch of the eyes, it is hidden in the far greater influence of other factors. The eyes would undoubtedly elongate permanently were the pins fitted perfectly tight.

In like manner, the influence of the percentages of excess of the head is indeterminate. Bar No. 709, with excesses of 31 and 43%, gave the same result at each eye. The eight eyes of the four bars Nos. 760-3, which were made from one mill bar, gave the following elongations at 24 000 lb.:

Head.	Excess.	Elongation.
763 *B*	51%	0.062
762 *A*	52	0.090
763 *A*	55	0.033
761 *A*	56	0.085
761 *B*	59	0.037
762 *B*	59	0.039
760 *A*	62	0.060
760 *B*	66	0.048

In these and other tests, however, there appears to be an influence due to the excess of material at the end of the eyes, which would indicate that for the best results this excess should be limited.

In the Watertown tests quoted, and in the riveted links, Nos. 758-9, the frontal section in the first being 86%, and in the second 119%, of the body of the piece, the material at the end of the eye tended to buckle instead of stretching, as is the case with smaller percentages in the end of the head. The study of the present tests leads the writer to believe that a great stretch of the eyes before rupture, heretofore considered as showing a tough and tenacious material, is no more desirable than a tendency to buckle in front of the pin. The best proportions of head to resist the elongation of the eyes under the working strains cannot be decided by the present tests. It is thought probable that, for circular heads, 50% excess across the eye, thus making the end section 75% of the bar, would be a favorable proportion.

In a general examination of the tests it will be seen that the bars of the higher tensile strengths gave the better results.

Before the tests had gone very far, it was decided that the tensile

strength should be advanced, the percentages of the heads increased, and the pin clearances for the bridge bars limited to $\frac{3}{64}$ in.

While the pin clearances, excess of heads, tensile strength, and thickness of the bars, as affecting the tensile qualities, undoubtedly have some influence upon the stretch of the eyes, they do not give a full explanation.

The original bridge bars, being more than 50 ft. long, were cut in half and additional eyes made, so as to make two test bars. In one case, above mentioned, four test bars were made from one bridge bar.

The four heads of Bars Nos. 706 and 707 (same original bar) give stretches, at 24 000 lb. per sq. in., ranging from 0.135 to 0.266 in., or about as 1 to 2. That a long bar (more than 50 ft.), would differ in quality at the two extremes, does not explain the difference, for, on any assumption as to which two heads were made at the adjoining cut ends, there is still a minimum difference of stretch of 0.059 in.

Similarly, for Nos. 708 and 709 there is a range from 0.032 to 0.106 in. with a minimum difference for any two adjoining heads of 0.074 in.

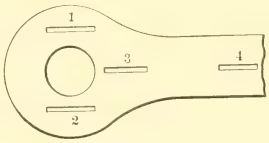


FIG. 5.

It will be noticed that the bars first tested gave the worst results (and it was fortunate that this was the case, for otherwise the necessity for a fuller series of tests might not have been recognized), presumptive evidence that more care had been taken for the later bars. The manufacturers and the inspectors assured the writer, however, that no change had been made in any of the processes of the manufacture of the later bars. It was then decided to cut samples from the two worst heads, Nos. 705 *B* and 706 *A*, and from one of the best, No. 711*A*, for nicked fractures and for tensile test, to see what the difference would be.

Samples Nos. 1 and 2 were cut from each side of the head, No. 3 from the neck, and No. 4 from the body, of the bar, as shown by Fig. 5.

The nicked fractures of the samples cut from Nos. 705 and 706 showed the same uniform fine granular structure with a clear bright luster. No difference could be detected between the several samples.

For No. 711, the samples showed a slightly coarser grain, with a suspicion of yellowish tinge in the samples from the head and neck.

These samples were examined, while fresh, by practical steel workers and experts. All agreed that they indicated nothing which would explain the different action of these heads.

The tensile tests (samples unannealed) are shown in Table 6.

TABLE 6. TENSILE TESTS.

Head. No.	No. of Sample.	Ultimate Strength.	Elongation in 8 in.	Reduction of area.	Fracture.
705 *B*.....	1	56 730	26%	52.8%	Silky, cup.
	2	60 870	25	53.8	" angular.
	3	59 260	26	56.5	" cup.
	4	61 680	28.2	56.6	" angular.
706 *A*.....	1	62 400	23	51.9	Silky, angular.
	2	62 750	24.5	48.8	" "
	3	60 060	25.5	59.1	" "
	4	75 760	9.5	50.6	" "
711 *A*.....	1	64 800	23.2	55.7	Silky, angular.
	2	64 960	19.5	51.7	" half cup.
	3	69 900	26	50.2	" angular.
	4	80 760	8	46.7	" cup.
Another sample cut from the body of bar No. 711, annealed.	5	60 430	25.8	55.8	Silky.

NOTE: As Bar No. 705 broke at a flaw in the head, *B*, the full tensile strength of the bar was not developed.

An examination of the tests for the riveted links, Nos. 758 and 759, shows that, even here, where there were no heat treatments, either of forging or annealing, the stretch of the eyes varied, the two eyes of No. 759 varying at each strain, and at 24 000 lb. the difference was 0.038 in.

An inspection of Figs. 1 and 2 shows that each class of bars, after giving a steadily increasing stretch, up to a certain point for each class, then begins to yield more and more rapidly, the bars of the higher tensile strength, as a rule, and presumably the harder bars, resist this breaking down up to a higher point. The unequal pulling of the metal in different heads and in different parts of the

same head, as shown in Table 5 and Fig. 3, shows that the metal is not homogeneous, but is softer in some bars and in different parts of the same bar. It is probable that the breaking down of the metal in front of the pin unequally is a large factor in the problem.

It is undoubtedly a great mistake to seek a soft and ductile eye-bar by using either low tensile material or softening processes.

To get the best results, we must either use steel of a higher grade, or else stretch the eyes longitudinally, cold, before the final boring to exact length, or do both. The writer believes that, with proper appliances, eye-bars can be made, which will not stretch in the eyes within the maximum working strains, without greatly increasing their cost.

As the tests here recorded have extended over a year's time, and every effort has been made to have them fairly represent the bars as manufactured from time to time, it is believed that the actual bridge bars will be better than those tested, the pin clearances and proportions of the heads being better, and the tensile strength somewhat higher.

These tests do not take any account of the elastic elongation of the eyes, which no doubt occurs, but would probably be small and constant for the different bars.

Practical Consideration and Application of the Results of the Tests.

Upon the development of the fact that eye-bars were not perfectly elastic, even at low working strains, and took an increasing permanent stretch with increasing loads, it became a grave question as to "How will a series of such bars pull together?"

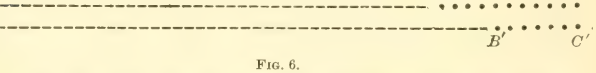


Fig. 6.

If two or more bars with different curves of stretch are strained to a fixed amount on the same pins, the parallelism of which is assured, the total elongation, $A\ C$, in Fig. 6, would be equal, but the permanent elongations, $B\ C$, being different, the elastic elongations, $A\ B$, must have a like difference.

The elastic elongations, $A\ B + A'\ B'$, corresponding to the total

load, must be divided between the two bars proportionately to $A\ B$ and $A'\ B'$. The difference of strain on the two bars will depend upon the ratio of the difference of permanent stretches, $B\ B'$, to the average elastic elongation, $A\ B + A'\ B' \div 2$.

The permanent stretch, at any working strain, being independent of the length of the bar, while the elastic elongation is proportionate to the length of the bar, the difference of strain in two such bars will be less as the lengths of the bars are greater.

In Fig. 2 the line, $A\ B$, is the elastic line for a bar 50 ft. long. The vertical ordinates between $A\ B$ and $A\ C$ give the elastic elongations of a bar of this length for any strain per square inch. The vertical distances from $A\ C$ to the curve of any bar gives its permanent stretch for each strain. Any line, $A'\ B'$, drawn through the curves of stretch of the several bars parallel to $A\ B$, will give, at the points of crossing, the strain on each bar for that condition of loading. The total elongations, being between two parallel lines, must be equal at these points.

For bars of other lengths, the elastic line must be changed to suit each particular length.

Although it is believed that the actual bridge bars are better than the bars tested, it will be assumed that the various bars shown in Fig. 2 cover the extremes, and represent the variety of bars to be used.

By drawing any line, $A'\ B'$, parallel to the elastic line for a bar 50 ft. long, and taking off the strain on each set of bars, we can readily get the average strain for all the bars and the limits of the variation, for example:

Bar.	No.	Strain.	Sum.
1	706	17 600	17 600
2	705–7	19 200	38 400
1	709	19 750	19 700
12	760, etc.	20 600	247 200
3	711 "	21 600	64 800
5	710 "	21 600	108 000
3	712 "	21 800	65 400
4	713 "	22 450	89 800
—		—	—
31	Average,	21 000	650 900

Which indicates that for bars of this kind, when pulled together on pins held parallel, for an average working strain of 21 000 lb. per sq. in., the softest bar will have only 17 600 lb. per sq. in., or about 84% of the average strain; and the hardest bar will have 22 450 lb. per sq. in., or about 107% of the average—strains not disproportionate to the capabilities of the different kinds of bars.

For the longer bars, up to 58 ft.—50 ft. being the minimum length—the difference in strain will be still less.

For bars of short lengths, under high working strains, the difference in strain becomes very great, which renders the use of such bars very undesirable. It will be seen that, for much lower working strains and short bars, the bars will be subject to a like variation of strain, with long bars and the higher strains.

It is very sure, therefore, that, when using high working strains, as are required for structures of great magnitude, long bars only must be used, if this stretch of the eyes cannot be overcome.

Camber.

For the working strain of 21 000 lb. per sq. in., it was found that a full set of bars of this kind would take a permanent elongation of about $\frac{1}{16}$ in., or $\frac{1}{32}$ in. for each eye, and this amount was provided for in all camber determinations. Further, it is thought that the probable error at the center of the channel span will not be more than $1\frac{1}{2}$ in. either way, an amount of no importance.

There are other features, connected with the action of such bars, which have been considered and provided for, but they do not come within the scope of the present paper.

DISCUSSION.

Mr. Seaman. HENRY B. SEAMAN, M. AM. SOC. C. E.—The matter of chief interest in connection with these full-sized tests is that, on 50-ft. bars, they offer a confirmation of the specimen tests made by Bauschinger years ago. Mr. Cooper's tests on eye-bars show a permanent set at a strain of about 12 000 lb. per sq. in., while his specimen tests indicated an elastic limit of about 35 000 lb. If his eye-bars had been still longer, it is possible that a permanent set would have been observed at even a lower strain. This, to the speaker's mind, is a very satisfactory confirmation of the deduction of Bauschinger that, after a strain is once applied, the elongation is never entirely eliminated, although it may gradually decrease if allowed time for rest.

These tests, the speaker believes, confirm the statement made by him in a paper* upon the Launhardt formula, that the experiments of Wöhler entirely destroy the theory of the perfect elasticity of metals as formerly accepted, and require that the term be abandoned and a new definition sought. Since that date the term, "yield point," has gradually replaced the old term, "elastic limit."

Mr. Cooper's experiments are valuable in demonstrating the necessity of considering the results of refined testing in large structures.

Mr. Merriman. MANSFIELD MERRIMAN, M. AM. SOC. C. E.—The full-sized drawings exhibited by the author show the distortions in the eye-bar heads more clearly than the speaker has heretofore seen. From these lines it is possible to study the actual distribution of the stresses throughout the metal, and probably a more precise knowledge might be obtained than that which we now possess. The lines, ruled on the head before making the test, were parallel and normal to the length of the bar forming 2-in. squares, and the distortions of these squares indicate the nature and the relative intensities of the stresses. Where a square is seen to be distorted into a rectangle, one side being shorter and the other longer than 2 in., it is known that there existed compressive and tensile stresses at right angles to each other. Where a square is distorted into a rhombus, it is known that shearing stresses also prevailed. The speaker regards these drawings as of much interest and value, and hopes that the author may be able to publish one or more of them for the benefit of the engineering profession.

Referring now to the general question brought forward by the author, it seems to be proved by the tests that the elastic limit of the eye-bar, as a whole, is reached before that of the bar proper. It is not difficult to see that this is entirely due to the high com-

* *Transactions*, Am. Soc. C. E., Vol. XLI, pp. 141 and 146.

Mr. Merriman. pressive stress in the eye-bar head at the back of the pin, this being due to the small bearing surface between the pin and the head. The usual rule for determining the bearing compressive stress, by dividing the total tensile load by the diametral area of the pin-hole, is, of course, a rough approximation, and it is certain that, with the usual clearances, the actual stress between the pin and the eye-bar head is very much greater than given by this rule. As a consequence, the compressive elastic limit of the metal in the head is exceeded before the tensile elastic limit of the metal in the bar proper is reached. Fortunately, the shape and size of the eye-bar heads have been so proportioned by experiment that rupture almost always occurs in the bar, and hence the author's conclusions throw no distrust upon the eye-bar system of bridges, as far as safety is concerned. His investigation, however, is of value and importance in computing the camber of long spans, and also for cases where the deflections of the ends of projecting trusses require to be computed.

In order to decrease the compressive stress in the metal back of the pin, it has been suggested to increase the thickness of the head of the eye-bar, and also to use a harder steel for the head. While the head can be made thicker, it is doubtful if it would be expedient to do so with such large eye-bars as those used in the Quebec Bridge. The use of harder steel for the head does not seem practicable unless the bar itself is also of the same grade of steel; in this case the elastic limits of both head and bar would be higher, the allowable unit stresses would also be taken higher, and, hence, the same phenomena as before would occur.

A third method that may be suggested is to cut the eye-bar hole of oval shape, the shorter diameter of the oval being a little larger than the diameter of the pin, while the longer diameter, which is parallel to the axis of the bar, is sufficiently large to give ample clearance. The curvature of the oval at the back of the pin should be greater than that of the pin, so that the pin, when first brought into bearing, does not quite touch the back surface of the hole, but bears along the head at two places on each side. The curve to be used should be such that, for a certain tensile stress in the bar, say, 15 000 lb. per sq. in., the radial compressive stresses between the pin and the head would be closely equal over an arc of 120°; if this can be attained, the intensity of the radial compressive stress will be less than 18 000 lb. per sq. in. The theoretic determination of this curve is not an easy matter, for the pin, also, is deformed as the stress increases, but a few experiments would undoubtedly result in producing an oval hole for which the distortions of the head would be very much less than those shown in the author's drawings. While the cutting of such holes would add somewhat to the cost of the eye-bars, it may be noted that the difficulty of inserting a pin through

many bars in erection would be much diminished, since the oval holes would furnish ample longitudinal clearance. Mr. Merriman.

ALBERT J. HIMES, M. AM. SOC. C. E. (by letter).—That an eye-bar is known to be permanently elongated in the pin-hole when the strain in the body is not more than 12 000 lb. per sq. in. is a startling fact, and should have been discovered before. In now bringing the matter before the Society, Mr. Cooper has added one more important service to the generous list which he has already given to the profession. Mr. Himes.

Although it is not found in practice that bridges are developing a deflection such as would be caused by elongation of the pin-hole, and, in taking down numerous old bridges which have been subjected to loads far beyond those contemplated in their design, no deformation of the pin-hole has been noticed, these facts merely demonstrate, as in the case where one of a pair of eye-bars carries the whole load, that the assumptions of loading and factor of safety are so liberal that defects, like this lack of strength in the pin-holes, have not produced any noticeable effect in working structures.

Such defects, however, are elements of weakness which greatly reduce the supposed factor of safety and render of small value the liberal sections brought into use by some of the impact formulas.

The author's discovery will also do much good indirectly by checking a tendency toward over-confidence in the perfection of the art of bridge building. That there is still something to discover is very evident, and the need of greater caution is plainly indicated.

A theoretical discussion of the effects on the pin-hole of tension in the bar gives results so much in accordance with those described by the author that it will be presented for consideration.

If an eye-bar be imagined to be divided longitudinally on the center line, and each half of the bar to carry its proportion of the tension independently of the other half, and then, if the head of the bar be cut through the center of the pin-hole at right angles to the axis of the bar, the result is a free body, shown in Fig. 7. This free body is acted upon by only two forces: tension in the body of the bar and a parallel tension in the head; but the two forces are separated by a distance, a, thus forming a couple. The moment of this couple must be resisted by a section of the head, $A\ B$, and, in a specific case, the computation of the outer fiber stresses in this section will show that the usual working stress in the body of the bar produces stresses in the section which exceed the elastic limit.

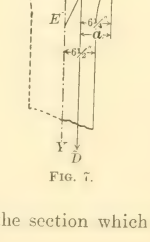


FIG. 7.

Mr. Himes.

Assume a bar 1 in. thick.
Assume a unit stress of 12 000 lb.
Tension at $D = 6\frac{1}{2} \times 12\,000 = 75\,000$ lb.
" " $C = 75\,000$ lb.
Arm of couple $= 6\frac{1}{4}$ in.
Moment of couple $= 469\,000$ in-lb.
Section $A\ B = 7\frac{1}{2} \times 1$ in. $= 7\frac{1}{2}$ sq. in.
Moment of inertia of $A\ B = 35.2$.

$$\text{Outer fiber stress at } A = \frac{M\ e}{I} = \frac{469\,000 \times 3.75}{35.2} = 49\,960 \text{ lb.},$$

which exceeds the elastic limit.

This condition agrees precisely with those reported by the author. He discovered compression at B and elongation at A, and a permanent deformation when the unit tension at D was 12 000 lb.

With the change of shape of the pin-hole, there must come a re-distribution of stress in the section, $A\ B$, so that the tension at A will decrease and the compression at B will be changed to tension. If the direction of the tension at C be changed so as to pass through the head and intersect the axis of the bar at E, there is no longer a couple, and the tendency to deformation which it produced in the section, $A\ B$, has ceased; or, it may be said that a second couple has been formed by the lateral pressure against the pin and its corresponding resistance in the section, $X\ Y$, this couple acting in a direction opposed to the first couple, and therefore relieving the bending stress in the section, $A\ B$. This condition agrees well with the fact that after a slight stretch at A, the pin-hole is not generally ruptured, although the bar breaks in the body under a tension four or five times as great as that which caused the first deformation in the pin-hole.

Mr. Cooper discovered that a slight variation in the pin-hole clearance produced no appreciable effect in the deformation of the bar, and this fact also agrees with the theory, since a maximum variation of, say, $\frac{1}{16}$ in. is very small, compared with the arm of the couple.

In attempting to meet the requirement that bars tested to destruction shall break in the body rather than in the head, the manufacturers, apparently, have rested content with their success, and have paid little attention to the character of the deformation.

If the section, $A\ B$, could be given a moment of resistance great enough to keep the tension at A well below the elastic limit, a condition which exists in the riveted flat bar, Fig. 4, the defect would be corrected. Another remedy would be to alter the shape of the pin-hole and bead to conform approximately to that due to final distortion. The latter method would not be perfect, but it would greatly lessen the defect. The bar would still have to stretch enough

to come to a bearing on the sides of the pin, after which it might be fairly assumed that the bending moment has been eliminated. The manufacturers would find some difficulty in making the elongated holes, but the tests appear to indicate that an improvement is needed. Mr. Himes.

The author's conclusion, that bars of high tensile strength are to be preferred because they exhibit less deformation in the tests, would seem to be unsound, because such bars would be strained to the yield point in the section, *A B*, as well as bars of softer material, and, if steel must be deformed, it is well known that the softer grades are safer.

While eye-bars are under consideration, the writer desires to say something in reference to annealing. There seems to be a prevalent idea that the full-sized test is satisfactory if the bar does not break in the head. That result is assumed to prove the success of the bridge shop; and previous specimen tests have shown the character of the mill product.

Fractures partially crystalline are very common, and brittleness sometimes appears. These defects, in all probability, are due to heat treatment, and, as the bars have been annealed, they cannot be charged to the rolling mill. The full-sized test should be a test of annealing as well as a test of the workmanship on the heads, and there can be no true test of annealing unless a bar is broken from every charge of the annealing furnace.

The annealing of eye-bars has long been subject to the personal skill and supposed infallibility of men who, though faithful and skilful beyond the average, have, nevertheless, a poor conception of the scientific properties of steel. The importance, in annealing, of a uniform and rapid heating to a temperature, not too high, and of uniform and fairly rapid cooling, is not generally understood. Ridsdale has shown the effects of too high a temperature and of chilling,* and bars that bore all the evidence of such treatment have been seen by the writer. It would seem that the substitution of a pyrometer for the time-honored color test would afford a more certain control of the temperature and be another step in the march of progress.

A. W. Carpenter, Assoc. M. Am. Soc. C. E. (by letter).—The author states that the failure of the usual assumptions, as shown by his investigation, is not of much importance in ordinary bridges on account of low unit stresses. It would seem to the writer that the stresses in ordinary bridges are frequently, if not generally, high enough to come within the range of those which are shown to be serious. With the increase of loads and with the impact, in the case of railroad bridges, the nominal stresses for which the structures are designed are greatly increased, and the details should be Mr. Carpenter.

* *Engineering News*, Vol. 46, pp. 238 and 276.

Mr. Carpenter. such as to take care of any increase in the stresses as well as in the main sections.

In view of the results obtained by the author, the present generally-adopted design of eye-bar heads is defective, even for ordinary structures, especially as the tendency is toward smaller heads. The author's experiments were conducted upon bars with heads larger in proportion than are now furnished in ordinary practice. The excess percentage through the eyes of the bars tested is shown to vary from 39 to 69%, with one exception, in which the excess was 31 per cent. The largest bridge concern in the country has, for a standard, a head with 30% excess of section through the eye, and guarantees the development of the full strength of the bars with such heads. It would appear from the author's tests that the stretch of the pin-holes in such heads, due to stresses within working limits, would obtain in greater degree than with the larger heads. It would seem, therefore, that the manufacturers should change their standards to produce larger heads, even at the expense of some metal and room for clearance. This would seem to be a primary step in the right direction. As pointed out by the author, however, something more is necessary. He shows that reducing the pin clearances and increasing the size of the pins does not affect the results.

He calls attention to the superiority of harder steel, and the tests appear to confirm this superiority. The writer believes that the steel used for eye-bars and other annealed members should be of a harder grade than that used for unannealed material, so that the finished work may be more nearly of the same strength throughout. With the added advantage of stiffening the pin-holes, this would surely seem to be the proper selection of material. This, of course, is in line with the author's suggestion, his other recommendation being to stretch the eyes longitudinally before final boring. The latter may be practicable, but seems to be a little doubtful.

The writer would offer the following suggestion: that the heads of the bars be made thicker than the bodies, a method which was extensively used at one time. This would seem to be the most efficient method of decreasing the maximum pressure of the pins on the pin-holes. This pressure, owing to the necessary clearance of the pin, however infinitesimal, must be a variable pressure, having a maximum at the back of the pin-hole in the line of stress. This maximum pressure is reduced directly in proportion as the head is thickened. The section in the head could thus be very rapidly increased without increasing the diameter, and the manufacture of such heads would seem to present no difficulties. The disadvantages would be in the increased space occupied in packing, the increased length, and, probably, in the increased strength required for the pins. A minor advantage in the thickening of the heads would be the

greater separation of the bodies of the bars, as these sometimes lie too close for painting. It is possible that a very slight thickening would give the desired result, but this is a matter which it would only seem possible to prove by experiment. Mr. Carpenter.

An old handbook of the Phœnix Iron Company gives a table of dimensions of thickened eye-bar heads, which shows the range of thickness varying from $\frac{1}{4}$ to $\frac{5}{8}$ in. for bars up to 6 in. in width. The excess section obtained varies from 43 to 87 per cent. The material, of course, was iron, and the writer understands that the heads were formed partially by piling and welding, and partially by upsetting.

The writer's suggestions, summarized, would be, to make some bars with heads of the usual circular shape, and with a section through the pin-hole 50% in excess of the body of the bar, using medium steel running to the highest limit of tensile strength (70 000 lb. ultimate strength), with heads thickened, say, 25% over the body of the bar, and test these for the stretch of the pin-holes on the lines followed by the author. Some change in the ordinary design and method of manufacture should be made to remedy the defect pointed out.

JOHN THOMSON, M. AM. SOC. C. E. (by letter).—The following observations, while not derived from a line of application similar to that described by Mr. Cooper, may yet have some interest, and indicate a line of further experimentation which, if carried out properly, may cast additional light upon the subject. Mr. Thomson.

As to the statement:

"We have assumed that a set of bars carefully bored to an exact length would all pull to an equal strain, as long as the elastic limit measured on the body of the bar was not exceeded."

The writer can say that, in his experience with short connecting rods, used for heavy duty on printing and embossing machinery, it has been known, for a considerable time, that the design of the eyes and the relative proportion existing between the bearing surfaces thereof and the pins upon which they act, are factors of the first importance.

Thus, if the eye-bars described in the paper are regarded as connecting rods to be used in tension on a machine, then, in the writer's opinion, the reason they failed in the manner set forth would be due to the fact that the bearing surfaces, as between the bores of the eyes and the pin, have not sufficient area.

Fig. 8 is a view of a 15-in. bar, 2 in. thick, with a 12-in. pin. The effective arc of contact on such an eye and pin will not exceed 120°; if loosely fitted, as stated, it will hardly exceed, say, 90°, which is the arc of contact in primary intimate contact. But, assuming the maximum, or 120°, the effective area in contact to resist the pull of the bar will be approximately 25 sq. in. The area of the rod,

Mr. Thomson. $15 \times 2 = 30$ sq. in., which, when subjected to a stress of 24 000 lb. per sq. in., gives a total test load of 720 000 lb.; and this, divided by the area of the bearing, gives an average pressure of 28 800 lb. per sq. in. of that surface, or 4 800 lb. per sq. in. in excess of the tensile stress per square inch in the main body of the eye-bar. The elastic limit of the steel is not given in the paper, but it may be assumed as being not far from the pressure developed within the eye upon the pin at the test-strain quoted. Be this as it may, it is a fact that, with the relative proportions adopted, the intensity of pressure, even upon the most favorable assumption of conditions, is greatest where it should be the least.

The remedy, assuming that the adopted cross-sectional area of the main body of the bar is essential, is to increase the area of the

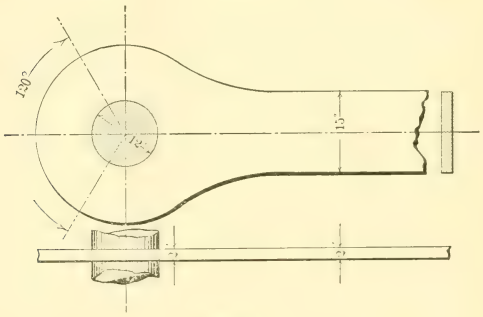


FIG. 8.

surfaces in contact between the inner surface of the eye and the bearing pin, and, in the writer's opinion, for such a purpose, the extent of this bearing should be approximately twice that of the cross-sectional area of the main body of the bar. This can be obtained in two ways: First, by considerably increasing the diameter of the pin and eye; or, second, by increasing the thickness of the eye. The latter method is regarded as more preferable. Such a construction is shown in Fig. 9, the outside diameter of the eye being decreased and its thickness doubled. The mass of metal is approximately the same in either instance. In this way the effective bearing surface is doubled, that is, it is 50 sq. in.; and, under a loading similar to that cited, the pressure per square inch would be 14 400 lb., or 9 600 lb. per sq. in. less than the tensile stress in the main body of the bar and 14 400 lb. per sq. in. less than that in the bar of Fig. 8. In the writer's judgment, this feature is the key to the problem. It

may not be quite so "handy" for rolling-mills to slab out bars of the form indicated in Fig. 9, but this would probably be "all to the good," as there appears to be no reason why bars of the dimensions given in the paper should not have their eyes formed by forging, or hydraulic pressure, in forming-dies, as has been done most successfully, in thousands of instances, in the writer's own experience. In this way, too, there is another advantage in that the forged bore of the eye can be swaged, relatively cold, thus condensing and hardening the metal where it bears upon the pin. Mr. Thomson.

There is another point relative to this matter, which, however, is presented with some hesitancy. It is illustrated by Fig. 10. Here, the query is: Would it, or would it not, be advantageous to flatten the pin at the top and bottom, line *S*, at right angles to the line of strain, *P*? As to whether or not this detail is new, the writer does

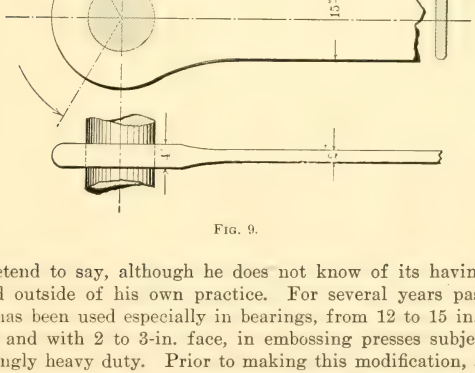


FIG. 9.

not pretend to say, although he does not know of its having been adopted outside of his own practice. For several years past, this detail has been used especially in bearings, from 12 to 15 in. in diameter and with 2 to 3-in. face, in embossing presses subjected to exceedingly heavy duty. Prior to making this modification, a number of these rods had failed, having fractured through the forward quadrants of the eyes, where, it may be observed, practically all such fractures take place, at least in the service now being considered. Since making the change in the bearings, that is, planing the flats, *C*, at the top and bottom of the journal or pin, not a single eye has parted, although the duty demanded has since been considerably increased. What is the reason? This the writer does not pretend to answer definitely as the result of actual demonstration, that is,

Mr. Thomson. demonstration undertaken for the express purpose of proof, but his theory as to the cause may be stated as follows:

When the eye of the rod is subjected to such a stress that it is stretched away from the free side of the bearing, as h, or, what amounts to the same thing, if the forward bearing surfaces wear or yield under compression, this produces motion at the top and bottom of the journal or bearing-pin; and, as a considerable portion of the bearing, in these locations (20°, 30°, 40°), is but slightly effective in resisting strain directly, applied as at P, these segments act as highly

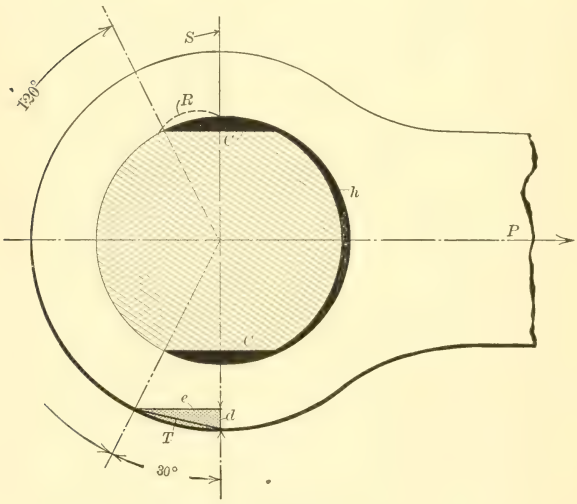


FIG. 10.

effective wedges, to augment the direct or normal strain, and operate to burst the forward quadrants of the eye. This so-called "wedge" is denoted, on the lower edge of Fig. 10, on an arc of 30°, in which the bursting effect would be as the relation of the versed sine, d, to the sine, e; or, say, about five times that of the primary strain, friction being disregarded. Obviously, the same result would be obtained by slightly clearing the eye, as at R, or by a less flattening of the pin, as denoted by the line, T. It may be mentioned that these clearances, in a revolving bearing, afford excellent means for lubrication, and permit a preliminary fit, upon the circular arcs, of

the journal or pin, considerably closer than would otherwise be permissible. Mr. Thomson.

Whether the foregoing theoretical explanation stands or falls, the proof of the effectiveness of the principle in practical use, in the application cited, is complete; and the writer would have no hesitation in utilizing it under any analogous condition. In other words, paraphrasing a portion of Mr. Cooper's opening text, "hold fast that which is good," whether or not one finds theories to fit the case. This, however, is not intended to mean that it is not somewhat better to have a close working combination between theory and practice, which is intended to apply especially to Figs. 8 and 9 and the description relative thereto.

Mace Moulton, M. Am. Soc. C. E.—The author has stated that, under ordinary working stresses, the elongations in the shorter bars differ from those in the longer bars. Does it follow, therefore, that the general methods of computing the deflections will have to be modified on account of elements introduced by the difference in the lengths of the bars? This might occur, for example, in the case of a cantilever in which the top slopes toward the ends of the truss. Mr. Moulton.

In the ordinary method of computing deflections, this difference in the lengths of the bars is usually taken into account, but, from the author's statement, it would seem that the deductions would be necessarily different in the case of bars of different lengths. As a result of the author's tests, is it possible to compute approximately how much allowance should be made?

John D. Van Buren, M. Am. Soc. C. E. (by letter).—While reading Mr. Cooper's valuable paper, it occurred to the writer that a rubber eye-bar would act in very nearly the same manner as a steel one, within the limits of elasticity, and would show the strains plainly on a small scale. The experiment was made, and the results illustrated by Fig. 11 are submitted at the suggestion of the author, who informs the writer that the lines of the rubber eye-bar correspond exactly in character with those of the large steel bars of his own experiment. The writer is in hopes that a more elaborate experiment with rubber may lead to results of some practical importance in determining the distribution of the stress quantitatively as well as qualitatively. Mr. Van Buren.

The dimensions of the eye-bar were as follows:

Neck $= 1\frac{3}{8}$ by $\frac{1}{2}$ in.;
Eye $= 2\frac{3}{4}$ in., outside diameter;
Pin $= 1$ in. diameter.

The bar, before the application of the stress, is shown in full lines, with squares inscribed on it. The bar, after the application of the stress, is shown in dotted lines. The pin-hole, as distorted

Mr. Van Buren. by the stress, is shown by the dotted line, *a b E d*. The slanting dotted lines show the directions of the strains or flow. The small dots show the corners of the squares after distortion by the stress.

The difference between the areas of the original and the distorted squares, or between their sides or diagonals, measures approximately the stress at any particular place; tensile if the distorted squares, or lines, are the greater; compressive if they are smaller, with intermediate shearing stress.

The following indications are noted: The maximum stress is near the pin, along the lines, *b E* and *d E*, and is excessive, while the stress on the outer edges, *D* 8 and *B* 8, is comparatively small. The cause of this is evidently the bending action on each side of the pin just below *B D*, which increases the tension at the pin and reduces it on the outer edges. At *a*, for about half way to *A*, there is compression, and for the remainder of the distance there is tension; so that there is a neutral point between *a* and *A*. The top of the eye above line 4, or line 5, appears to be strained somewhat in the manner of a beam. There is compression at *E*, where the two streams of the flow meet. There is apparently a curved boundary of shearing stress, starting near *b*, cutting *a A* below *A*, and ending near *d*, surrounding the compressed area.

The excessive stresses and strains in the steel eye-bars, at and near each side of the pin-holes, account for the permanent elongations of the pin-holes even under moderate stresses, discovered by the author.

With a solid having considerable compressibility and a characteristic texture, however, the experiment is not complete without the measurement of the distortions in thickness, that is, in a direction perpendicular to the face of the bar, or diagram. As rubber has very little compressibility—in other words, has a very large modulus of elasticity of volume—it is evident that the volumes of the prisms represented by the squares will remain nearly constant, and that, therefore, the changes in areas of the squares will be accompanied by proportional changes, of an opposite character, in the thickness. Practically, with rubber, the coefficient of transverse linear expansion or contraction must be one-half that in a longitudinal direction—that of the load. The relations between the stresses and strains, in such a case, are comparatively simple; but, with more compressible solids having complex textures, it is difficult, if not impossible, to formulate this relation, even with a complete record of the distortions in the three directions. While it may not be possible to determine the stresses from the strains by the distortions in area alone, these distortions furnish a safe guide to the practical experimenter in search of the best shape—which requires the greatest possible uniformity in the stresses and strains. Pro-

EXPERIMENT WITH RUBBER EYE-BAR

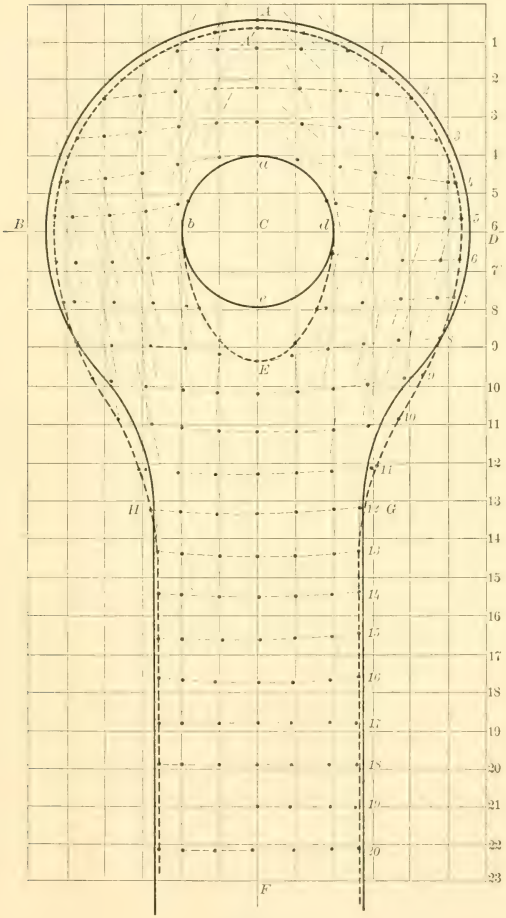


FIG. 11.

Mr. Van Buren. gressive tests, marking the points of set and rupture, as carried on by the author, are the only safe guides. The mathematical theory of elasticity applied to a diagram of strains in a solid of complex structure leads to nothing practical.

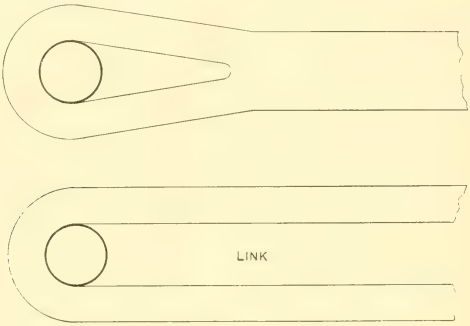


FIG. 12.

Aside from considerations relating to the difficulties of manufacture, the results of this little experiment seem to point to the modifications indicated by Fig. 12 as remedies for the excessive stresses adjoining the pin. The excess in these shapes could be considerably reduced.

Mr. Schaub. J. W. SCHAUB, M. AM. SOC. C. E. (by letter).—The results obtained by the author are not new. As he says, referring to his notes on the Eads Bridge, he finds, in pulling some iron eye-bars up to a proof stress of 18 000 lb. per sq. in., that a permanent deformation took place in the eyes. If the writer may be pardoned for the transgression, it may not be out of place to say that he believes the history of the building of the Eads Bridge to be the greatest educator in the art of bridge building, even to-day.

In those days it was customary to pull all eye-bars, in the shop, up to a proof stress, usually twice the working stress, or about 18 000 to 20 000 lb. per sq. in. This applied to all iron eye-bars, and was the accepted practice until the advent of the steel bar. This test was for the purpose of developing any flaws in the head which might exist in the weld; but, as far as the writer knows, it never developed anything, except that a permanent set was produced in the head of the bar. This fact was well known, and in looking over his notes on iron eye-bar tests made at Edge Moor, in the period from 1881 to 1883, the writer finds in many cases the note that a permanent set took place in the head of the bar. This

deficiency was first noted when the original mill scale and cinder began to flake from the head of the bar, sometimes back of the pin, but usually near the neck of the bar. The permanent set given to the bar was never as much as $\frac{1}{32}$ in., so that little or no attention was paid to this deficiency, and, as long as the bar did not ultimately fail in the head, it filled all the requirements. Mr. Schaub.

These distortions are not confined to eye-bars. They will be found in compression members, as well as in tension members, and in riveted connections as well as in pin connections; in fact, in all cases where the stress is applied to a theoretical point in the member, and where insufficient means are provided for distributing this stress into the body of the member. This defect is inherent in all designs, more or less, and cannot be avoided without providing a sufficient amount of extraneous material in the connections to distribute the stresses properly, within the limits of elasticity of the materials.

In the case of eye-bars, the difficulty can be overcome to a great extent by thickening the heads; and, at the same time, making the heads elliptical, or longer, so as to increase the metal in front and back of the pin. This was the shape of the heads made on iron bars, before the circular head came into general use, and should never have been abandoned.

The criticism offered by the author as to the present form of eye-bar is just, and the defect should be remedied.

H. de B. Parsons, M. Am. Soc. C. E. (by letter).—The author is to be congratulated for having brought out so clearly the distortion of an eye-bar end, and for the clear manner in which he has stated the result of his observations. Mr. Parsons.

Having examined the drawings of the eye-bar ends, both before and after the application of stress, the writer does not see why the comparison between the two forms (before and after stress) should be made from a line passing through the original center of the pin.

The pin pressed against the metal of the far end of the eye, and remained pressing against the metal at that point until the end of the test. Assuming that the pin remained stationary, the metal of the eye bearing against the far side of the pin on the axial line of the eye-bar also remained stationary; and whatever distortion was produced in the head by the stress was a movement from this point.

Instead of comparing the distortion along lines at right angles to themselves, and at right angles and parallel with the axial line of the eye-bar, the writer is of the opinion that the distortion is more accurately shown by lines drawn more nearly approaching the "lines of pull," namely, concentric around the far end of the

Mr. Parsons. pin and approximately parallel to the sides of the eye as they approach the main body of the bar.

These lines of pull were drawn by the writer on the drawing of the eye-bar before the application of stress and their position transposed by interpolation on the eye-bar after the application of stress. This is shown in Plate XXVIII.

The original lines drawn by the author at right angles to the axial line of the eye-bar were numbered, and the writer measured the distances on his lines of pull between these numbered lines. He also measured the corresponding distance on the eye-bar after the application of stress, and found the difference in length. This difference in length measured the distortion.

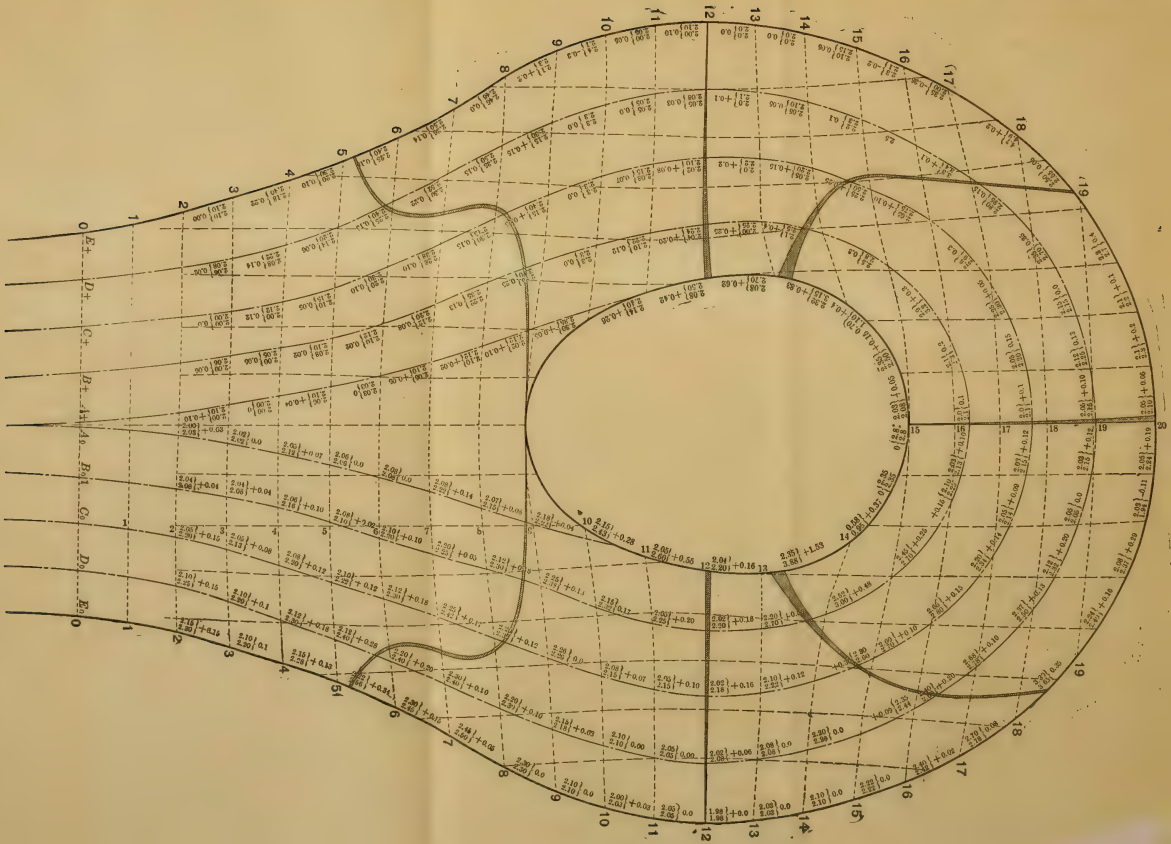
These distortions, expressed in percentages of the length of the measurements on the eye-bar before the application of stress, are laid off in Fig. 13, as ordinates from a line, the divisions of which represent the numbered lines drawn by the author; they correspond to the numbers shown in Plate XXVIII.

In Fig. 13 the curve, *A*, represents the distortions of the line of pull marked *A* in Plate XXVIII; the curves, *B*, *C*, *D* and *E*, represent the same for their respective lines of pull. The distortion was measured on both sides of the axial line, and the results are recorded in Fig. 13, one with a cross and one with a circle. The curves in Fig. 13 are drawn midway between these points so as to average the discrepancies caused by:

First.—The inequalities in the metal,
Second.—Any indirectness of pull, and
Third.—Any errors in tracing off the lines from the eye-bars as marked by the author.

In a few places, especially noticeable where the curve, *E*, passes below the horizontal line, the distance on the eye after the application of stress was less than the original distance, thereby making the distortion appear to be negative.

As these distances were measured along the lines of pull, this negative result is probably due to errors in tracing off the original lines from the distorted eye-bar, because the resistance of the eye-bar metal to distortion by compression is so much greater than its resistance to distortion by tension as to render this negative result to appear to be unlikely. The negative measurements mentioned in the paper were made at right angles, or approximately so, to the axial line of the bar, and these measurements would naturally become smaller as the pull stretched the metal and made it flow into a new form, thus diminishing the distances measured transversely, but increasing the distances as measured longitudinally or along the writer's lines of pull.



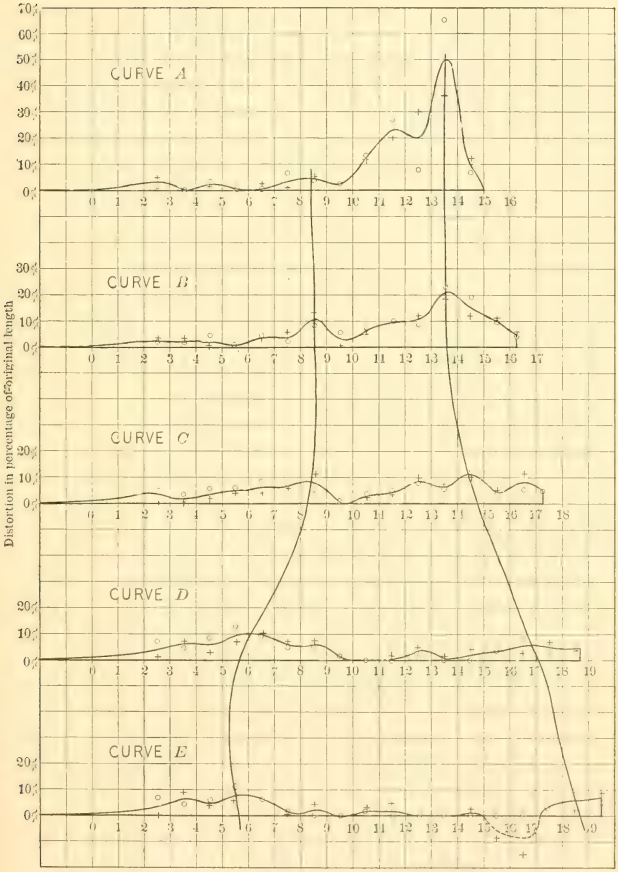

CURVE A
CURVE B
CURVE C
CURVE D
CURVE E
Distortion in percentage of original length

FIG. 13.

Mr. Parsons. From a study of the curves of Fig. 13, high points are noticed on each side of the center line through the pin. These points of maximum distortion are also drawn in Plate XXVIII, and are shown by the curved shaded lines. The first high point occurs on the outside of the eye, where the line of pull, *E*, makes the greatest angle with the axial line. The second high point occurs on the inside of the eye, where the round of the pin is acting like a wedge against the eye. On the axial line at the far side of the pin, the metal of the eye is in compression, and at the outside of the eye it is in tension.

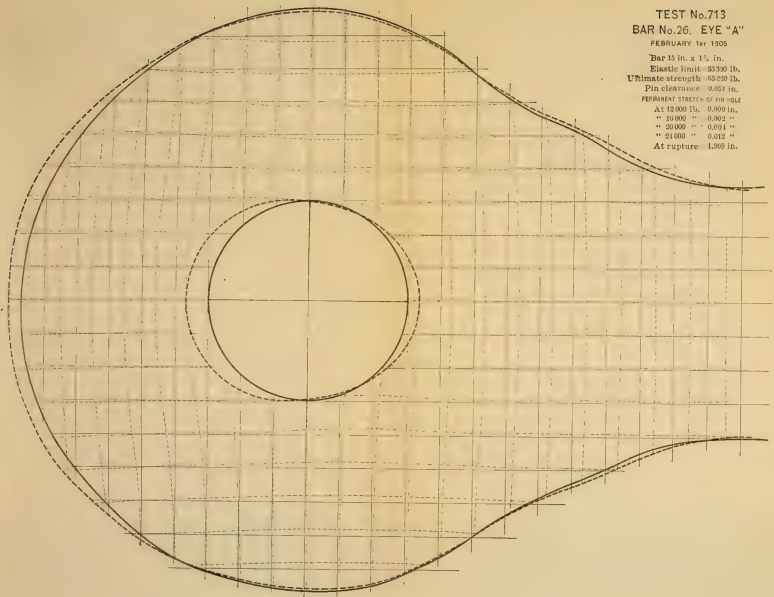
Mr. Cooper. THEODORE COOPER, M. AM. SOC. C. E. (by letter).—Since the presentation of the paper, four more bars have been tested for the elongation of the pin-holes. The results confirm the previous conclusions that, with the higher grade of steel now used, the elongations are more favorable than with softer material.

Test No.	Ultimate strength.	Elastic limit.	At 24 000 lb. elongation of pin-holes.	Elongation from out to out of pin-holes.
869	62 550	36 000	0.012	$\frac{3}{64}$
			0.032	
870	60 650	34 420	0.009	$\frac{5}{64}$
			0.061	
871	62 400	35 330	0.015	$\frac{3}{64}$
			0.012	
872	60 260	33 770	0.038	$\frac{6}{64}$
			0.047	

Tables 2 and 3 do not contain the recorded elastic limits of the bars tested. They are here given in order to fill out the record.

Test No.	Elastic limit.	Test No.	Elastic limit.
705	28 500	798	28 920
706	26 800	799	28 920
707	26 700	800	32 650
708	28 440	801	31 940
709	28 110	802	28 480
710	33 100	803	29 750
711	32 850	804	28 500
712	35 860	807	28 500
716	36 100	808	31 400
713	33 300	809	31 290
714	33 900	822	31 540
717	33 330	823	31 650
718	33 970		

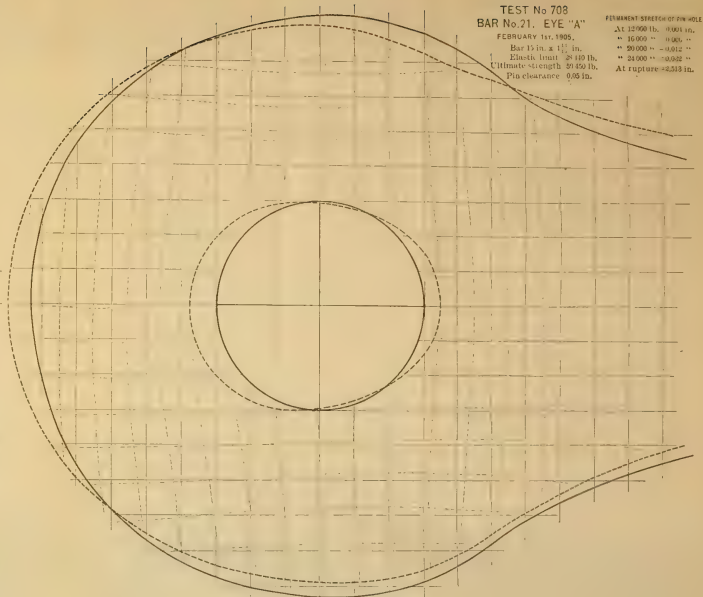
These are the elastic limits taken on the body of the bars. It will be noted that the bars giving the greatest elongations in the eyes, 705 to 709, all have low elastic limits.



TEST No. 713
BAR No. 26. EYE "A"
FEBRUARY 1st 1905

Bar 15 in. x 1¼ in.
Elastic limit 33 330 lb.
Ultimate strength 65 230 lb.
Pin clearance 0.031 in.

PERMANENT STRETCH OF PIN HOLE
At 12 000 lb. 0.000 in.
" 16 000 " 0.002 "
" 20 000 " 0.091 "
" 21 000 " 0.012 "
At rupture 1.969 in.



TEST No 708
BAR No. 21. EYE "A"
FEBRUARY 1st, 1905.

Bar 15 in. x 1¼ in.
Elastic limit 34 410 lb.
Ultimate strength 59 450 lb.
Pin clearance 0.05 in.

PERMANENT STRETCH OF PIN HOLE
At 12 000 lb. 0.001 in.
" 16 000 " 0.005 "
" 20 000 " 0.012 "
" 24 000 " 0.032 "
At rupture 2.313 in.

Mr. Cooper.

A desire having been expressed for the reproduction of the diagrams showing the scribed lines on the bars before and after testing, two typical cases are shown on Plate XXIX.

It appeared to the writer to give a better ocular representation of the movement of the metal to superimpose the two views so that the longitudinal and transverse lines through the center of the original pin-holes should coincide as nearly as might be. Mr. Parsons prefers to have the lines at the back of the pins as the reference lines.

Mr. Seaman has apparently assumed the set of the eyes to be the permanent elongation of the bars. The table of elastic limits shows that the bars did not take any permanent set at 12 000 lb. It is altogether probable that, when the body of the bar has as low a unit strain as 12 000 lb., some parts of the eye are strained beyond the elastic limit, and a permanent deformation of the eye takes place.

Mr. Thomson's account of the action of the connecting rods in heavy press work confirms a conclusion reached by the writer while studying these tests, that there must be cases in delicate mechanism where similar distortion would cause trouble. The remedy Mr. Thomson applied could not be used for bridge pins.

The rubber eye-bar test by Mr. Van Buren is very instructive in showing the same characteristic deformations as the steel eye-bars. While a better shape for the heads of eye-bars could undoubtedly be determined, the facility of manufacturing must be the governing factor.

For ordinary bars worked only up to the moderate unit strains of the usual specifications, the distortions of the eyes are not of serious moment; but for the greater structures of the future, where high unit strains are justified, it will be worth while to reduce or avoid the stretching of the eyes. The writer believes that it can be done without any important increase in the cost.

It will be seen from Tables 2 and 3 that, with the exception of the eyes of bars 705 to 707 (all soft bars), no eyes stretched more than $\frac{1}{8}$ in. under strains of 24 000 lb. on the body of the bars. For bars, therefore, which were not to be strained above this amount a longitudinal stretching of the eyes of $\frac{1}{8}$ in., if properly done, would probably make the bars capable of being worked up to 24 000 lb. per sq. in. without further stretching of the pin-holes. This could be done without the necessity of re-boring the pin-holes, except to correct the length of the bars. But, as this stretching should be done without straining the bodies of the bars, the slight change of length due to stretching the holes could probably be determined, after a few experiments, with sufficient closeness and allowed for in the first boring.

Mr. Cooper. As stated by Mr. Van Buren, "The mathematical theory of elasticity applied to a diagram of strains in a solid of complex structure leads to nothing practical." But still there is a certain mental satisfaction in forming a general conception as to the manner in which forces are transferred in such structures.

Let Fig. 14 represent an eye-bar end with a circular head cut transversely through the center of the pin-hole by the line, $A\ B$. The total force acting through the pin is equal to $2P$; the width of the bar equals n; with 50% excess, m equals $0.75\,n$; the thick-

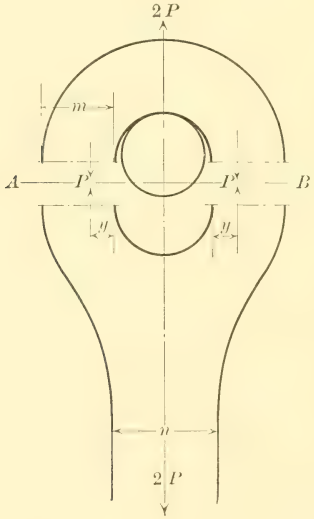


Fig. 14.

ness of the bar and head equals unity; the diameter of the pin equals $0.8\,n$.

Each side of the section will be acted upon by the forces, P and P, at points at the distance, y, from the pin-hole, so that there will be a moment, M, as well as a direct force, P, on each side of the pin-hole.

If the upper section of the eye be considered as a half ring acted upon by a force, 2 P, at the point of contact of the pin, the moments, M at A and B will be $M = 0.1817 \times 2Pr = 0.3634\ Pr$;

where r is the mean radius of the ring. The fiber strain will be $\pm$ 2.28 R, where R is the unit strain on the bar, n. For 10 000 lb. on the bar, the fiber strain will be $\pm$ 22 800 lb., and the direct strain will be — 6 667 lb., or there will be on the inside of the ring a tension of 29 467 lb. and on the outside a compression of 16 133 lb. Mr. Cooper.

This is entirely on the supposition of perfect elasticity, and that the force is applied at one point. As the fibers elongate and the pin gets greater bearing, the conditions change.

This illustration is given solely to show the general conditions of the strains, and the difficulty and uselessness of any full investigation by theory, when the assumptions have to be constantly changed for new and undetermined conditions.

When we consider the bending moments, which are so important as affecting the strength of the eye, it will be seen that thickening the eyes may not produce the best distribution of the metal.

In reply to Mr. Moulton, it may be said that, as the stretch of the eyes would be the same for long or short bars, the difference of strain on a set of short bars would be much greater, as the stretch would have a greater ratio to the elastic elongation. The writer believes that the averages he has reached from the series of tests submitted are sufficient to determine the camber allowances. He does not claim that they will apply to bars of other form, make or specifications.

The writer would refer those interested in this subject to his paper entitled "Observations on the Stresses Developed in Metallic Bars by Applied Forces."*

Mr. Himes says "if steel must be deformed, it is well known that the softer grades are safer." But if we do not want the steel to deform except elastically, the higher grades are better.

Experience for a number of years in the use of that grade of steel known as "medium" has shown that it is perfectly capable of standing without injury all the deformations of the manufacturing processes. The present tests have convinced the writer that such steel is also far better fitted to resist the stretching of the eyes in the finished structure than the lower grades.

In the struggle to get one uniform specification for structural steel, the low-grade steel manufacturers have succeeded for the present in lowering the standard by getting the most votes. There will be no difficulty, however, for the bridge engineer who desires the best material, to get it.

For the great structures of the future, the best material must be used, and any effort of the manufacturers to restrict structural steel to one grade, and that the lowest one, will be without success.

* *Transactions*, Am. Soc. C. E., Vol. VII, July, 1878, p. 174.

Mr. Cooper. The writer firmly believes that a point has been reached where there is a general acceptance of the future maximum loadings, the most suitable forms of structures for different cases, and the unit strains for railroad bridges.

The creative and experimental stage has passed. The many defects in design, proportions, forms of details, and of inferior materials can no longer be ignored. They have been heretofore hidden or unrecognized, as all deficiencies of past structures have been ascribed to the rapidly increasing train loads. Now that railroads are building their bridges for the greatest maximum loadings, it will not redound to the credit of the bridge engineer to accept inferior details, workmanship or materials.

It has been generally accepted that "medium" steel, ranging from 60 000 to 70 000 lb. ultimate strength, is about as high as we can go in "carbon" structural steel; that it can be readily obtained, and that, in proportion to its working capacity, it costs no more than the inferior article.

Of course, every engineer who has in contemplation structures of great magnitude is hoping for and seeking a higher grade of steel, the cost of which will not be prohibitory for the particular case. For general structural work and bridges, "medium" steel, owing to its cheapness and suitability, will maintain its supremacy.

In reference to his remarks about "pin clearances," "excess of head," etc., the writer did not intend to convey the idea that they were without influence on the stretch of the eyes, but that their influence was apparently so small when compared with the greater one of the material and the local strains, that it was unrecognizable.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1025.

ADDRESS
AT THE ANNUAL CONVENTION AT FRONTENAC, THOUSAND ISLANDS, N. Y., JUNE 26TH, 1906.

THE DEVELOPMENT OF WATER SUPPLIES AND WATER-SUPPLY ENGINEERING.

By Frederic P. Stearns, President, Am. Soc. C. E.

In selecting a topic for the address which the Constitution requires me to deliver on this occasion, I have found that there is ample precedent for speaking on the subject with which I am most familiar, namely, water supplies and water-supply engineering.

I was the more ready to choose this subject because it gives me an opportunity to speak from my personal experience of the advantages of co-operation between engineers and those engaged along other lines of applied science.

It would be futile in a short address to attempt to trace the beginning of water-supply engineering. That it was not in its infancy in the Roman era is a matter of common knowledge, both from the carefully written descriptions of the water-works of that era and from the remains now to be seen of the magnificent arches and other works near Rome and in former Roman provinces, which prove the great ability of the water-works engineers of that period.

Works for both public water supply and irrigation existed at a much earlier period in Greece, as is attested by Homer in the following description of the gardens of Alcinous:

"Two plenteous fountains the whole prospect crowned:
This through the gardens leads its streams around,
Visits each plant, and waters all the ground;
While that in pipes beneath the palace flows,
And thence its current on the town bestows;
To various use their various streams they bring,
The people one, and one supplies the king."*

Traces of water-works have been found near cities which flourished in earlier periods; and, as human nature and the human brain have changed but little since the beginning of written history, it can hardly be doubted that from the time when people first gathered together in large cities there has been some public method of providing the necessary water for drinking and other domestic purposes.

In the Middle Ages, and even in the period of the Renaissance, when the fine arts, literature and architecture flourished, the water-works engineer, if he existed, built few if any works worthy of especial notice.

Of what may be called modern water-works, London probably furnishes the earliest example. Before these were built there were, as is often the case, minor works for bringing water from springs near the city. One of them was completed in the year 1285, and others from time to time afterward, and there was obtained at a still later date a larger supply by pumping water with tidal power from the Thames at London Bridge.

The first extensive modern water supply, however, was that provided by the New River Water-Works of London, built by Sir Hugh Middleton, in the four years from 1609 to 1613. The water was obtained from springs near the River Lea, distant about 20 miles in a direct line from London, but the open canal through which the water flowed to the city made many windings to avoid hills and valleys, so that it had a total length of 38 miles. The canal had an inclination of 3 in. per mile; its width averaged 18 ft., and its depth seldom exceeded 5 ft. The springs failed to fur-

* Odyssey, Book VII.

nish the required quantity of water, and were supplemented by a direct connection with the river. These were very great works for those days, when, to quote words written in 1835, "the science and practice of civil engineering were very little known."

In 1633 London is said to have been well supplied with water from pipes in all streets, and in nearly every house the rent of which was from $75 to $100 per year.

The water companies had troubles then as well as now. The average remuneration for water was one farthing a barrel, but it is stated that "notwithstanding such palpable evidence of its cheapness, some persons have been loudly querulous concerning the high and unconscionable rate charged for water." Their complaints led to a parliamentary investigation.

It is also noted among the troubles of the period that persons living near the open aqueduct used it for bathing. Under the laws then in force they could be prosecuted only for trespassing, for which the penalty was transportation. Considerations of humanity are said to have prevented the water companies from prosecuting the offenders.

After London was well supplied with water, 163 years elapsed before a public water supply worthy of notice was provided for any of the larger cities of America. The first one to be supplied was Boston, into which water was introduced by a water company in 1796. The water was taken by gravity from Jamaica Pond, situated a short distance outside the city limits. Philadelphia built its own works, which were supplied with water by pumping from the Schuylkill River, and were first operated in 1801. New York was first furnished with a water supply by the Manhattan Company at about the same time. In all these cities the water was distributed through hollow logs, which are sometimes found well preserved at the present day.

The earlier water-works in America seem not to have been well appreciated, as the revenue derived from the Philadelphia water-works three years after the introduction of water was only $1 800 a year, and up to 1815 not enough had been realized from water rates in any one year to pay for the operation of the works.

The water supplies of New York and Boston continued to be wholly inadequate to the needs of those cities for many years, as

money was not plenty, and the cities were loath to embark upon great enterprises with which they were unfamiliar. After long delay, however, they proceeded to build permanent works on a most liberal scale. New York, especially, displayed great courage in deciding to build an aqueduct 40 miles long, having a capacity of 80 000 000 gal. per day, from the Croton River to the city. This work involved the great difficulty of conveying the water over the Harlem River at a height of more than 100 ft. above its surface.

Having once begun the construction of modern water-works, the rapid growth of American cities and the lavish use and waste of water have required a very rapid development of water-works systems in all parts of the country, and works of unprecedented magnitude are now being undertaken with less hesitation than the much smaller works of many years ago. The most notable of these recent undertakings is that of the City of New York for bringing water a distance of 100 miles from the Catskill Mountains, at an estimated cost of $162 000 000.

Let us now leave the development of water-works as a whole, and consider the development of their various parts and the influence which water-works engineering has had upon engineering along other lines.

The Romans were masters of the art of building masonry aqueduct bridges and aqueducts, and the advance upon their practice up to the present time is not very striking. We now build, however, aqueducts of much greater size, and have a knowledge of hydraulics and of levelling which enables us to produce much more efficient works.

The Romans excavated long tunnels—one of them is said to have had a length of 3 miles—through which to convey water, but these must have been especially difficult to excavate before the days of explosives.

Their greatest trouble was that due to the absence of suitable pipes for conveying the water across low lands, and it was this difficulty which caused the construction of the aqueduct bridges to which reference has already been made.

They had lead pipes, made by folding sheet lead and soldering the edges, and used tile and stone pipes in some instances, but none of them compared in efficiency with the modern iron pipe.

The date when iron pipes were first used for water-works is uncertain; there are instances of their use as far back as the middle of the eighteenth century, but they did not come into general use until after the beginning of the nineteenth century.

In 1746 the Chelsea Water Company, of London, laid a 12-in. flanged pipe to a reservoir at Hyde Park, which was taken up and relaid in 1791 "in consequence of its joints being perished." Another pipe laid a few years later in the Kensington Gardens was taken up in 1819 on account of incrustations on the interior surface of the pipe. A 5-in. iron pipe was laid to supply water to Edinburgh in 1787. The London companies changed from wooden pipes, which were in general use, to cast-iron pipes, between the years 1810 and 1820.

Mr. Thomas Simpson, the engineer of the Chelsea Water-Works, of London, designed the first bell and spigot pipes, about the year 1785, and at that time laid a few lengths of this pipe with lead joints as an experiment.

Philadelphia in 1804 laid about three-fourths of a mile of iron pipe as an experiment, and in 1817 began to use cast-iron pipes with bells, similar to those now in use. These pipes were imported from England.

It is of interest to note that as early as 1810 a 15-in. flexible cast-iron main with ball and socket joints at intervals was laid across the River Clyde to supply water to a portion of Glasgow. A trench was first dredged in the bed of the river, then the pipes, attached to wooden frames, were connected on land on one side of the river, and subsequently dragged across it until one end reached the opposite shore, the operation being aided by pontoons, much in the same manner that such work is done at the present time. The credit for the design of this siphon is due to the celebrated James Watt, the inventor of the steam engine, who, for some years, practiced civil engineering.

"A patent Steam Engine" was erected by the New River Water Company, of London, in 1787, and it is possible, though not probable, that this was the first application of steam for pumping a city water supply.

The steam engines first used for pumping water for Philadelphia, in 1801, were unique machines, as the walking-beams, fly-

wheels, connecting-rods, and even the cold-water pumps, were made of wood. As indicating the difficulty of constructing such machines in this country at that time, it is said that the main steam cylinder of one of the engines was cast in two parts, and that nearly four months were occupied in boring it out fit for use. The boilers were wooden boxes bolted and braced on the outside. "The fire-box inside the boiler was of wrought iron, with vertical flues of cast-iron."

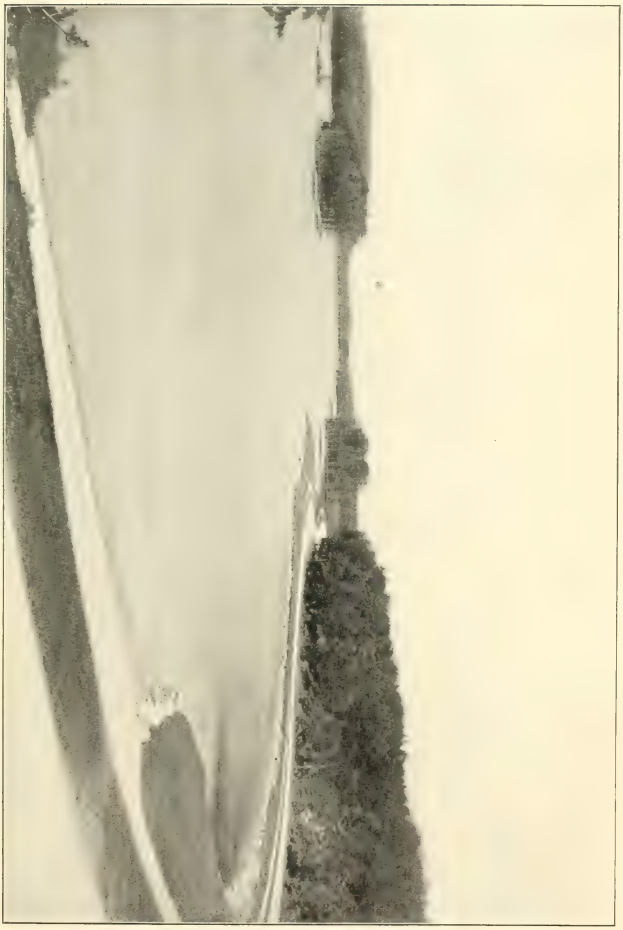
There has been a very great advance in steam-engine practice since those early days, and the water-works pumping engine, which has been developed by some of our most able mechanical engineers, has, much of the time, been in advance of other steam engines in efficient performance. It is worthy of note, however, that for a long time some of the Cornish engines, used in the mines in Cornwall, were the most efficient machines then in use, this result being obtained by a system of co-operation and nearly continuous tests, the results of which were published monthly.

The triple-expansion water-works pumping engine of the present day is a very perfect and smooth-running machine, which, with a high degree of economy, may lift a sufficient quantity of water to provide for the requirements of 300 000 people.

In the art of dam building the engineers of water supply and of irrigation have been rivals for many years. In India, and in recent years in the arid lands of our own country, the irrigation engineers have been designing and constructing great dams of masonry and of earth. Up to the present time, however, the water-supply engineers have created the greater works.

The New Croton Dam, completed within the current year, with a height from the lowest point of its foundation to its top of 297 ft., is the greatest of masonry dams, although the mass of masonry is small in comparison with that of the Great Pyramid.

The San Leandro Dam of the Contra Costa Water Company, which supplies Oakland and other cities near San Francisco with water, is the highest of existing earth dams. It is an interesting feature that this dam, as well as three notable dams connected with the San Francisco Water-Works—two of earth and one of concrete—were shaken by the earthquake of April 18th, 1906, when under full water pressure, without any serious result.



NORTHEASTERLY PART OF SPOT POND RESERVOIR.

Although the art of dam building has made such progress that great dams like those just mentioned have been designed and built of sufficient strength to withstand safely the tests of practical use, the theory of the design of both masonry and earth dams is only imperfectly understood. The rules given in the textbooks for designing masonry dams do not take account of some important forces affecting their stability, most of which are the result of temperature changes. Several of the more recent failures of masonry dams, most of them on southern rivers, could without doubt be traced to the neglect of the effect of forces which are not sufficiently considered in making designs.

Experiments made during the last twenty years to determine the laws governing the filtration of water through sand and soils of various kinds have furnished much additional information for use in designing earth dams, and it has been found that, with proper designs, earth containing a large percentage of fine particles may be substituted successfully for a masonry or clay puddle core-wall.

There are some water-supply features which are not creditable to the American people. First, the great waste of water which prevails in most of our cities, and second, the taking of water directly from polluted rivers without either filtration or long storage.

There is, among those who govern our large cities, an unreasoning opposition to the introduction of water meters, which are conceded by nearly all who know about such matters to be the most efficient means for diminishing the waste of water. The recent action by the government of one of our larger cities emphasizes this point. This attitude seems to have some relation to the commonly expressed view that water should be as free as air, and that on sanitary grounds it is not well to restrict its use, but no one has yet demonstrated the sanitary advantages of a leaky faucet or a defective ball-cock.

Experience shows that meters prevent waste, but do not prevent the use of as much water as is required for sanitation. Waste not only adds greatly to the difficulty and expense of obtaining a sufficient water supply, and requires the expenditure of money which might well be applied to improving the quality of the water by filtration or otherwise, but it adds greatly to the difficulty and ex-

pense of the sewerage system, especially where pumping and purification of the sewage are necessary.

There is undoubtedly a rapidly growing opinion among water-works engineers and those connected with water-works management that the general application of water meters is desirable, and the public and the governing bodies of our cities will in time become educated so that they will accept this view.

While I have said that the taking of water directly from polluted rivers is not creditable to the American people, I should add that under the leadership of the engineers and sanitarians of the present day this discreditable feature is fast disappearing. There is also to be said, as an excuse for the practice years ago when many such water systems were inaugurated, that the purity of water was then judged mainly from a chemical standpoint. Bacteriology was unknown, and the efficacy of filtration was not appreciated.

To show the point of view held thirty-three years ago, I quote from a paper written by one of the most prominent water-works engineers of that time:

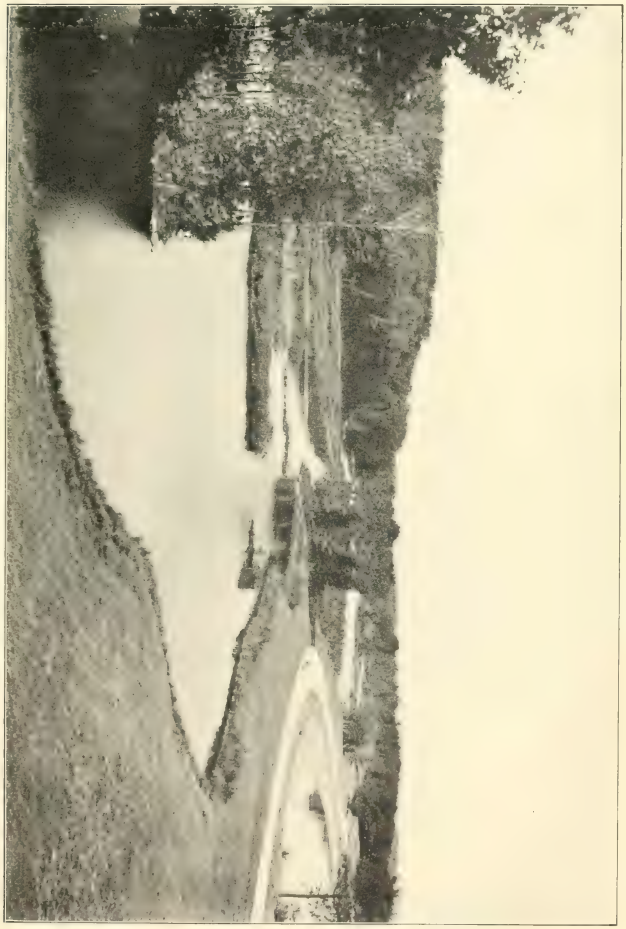
"The object of filtration is to remove the visible impurities of water and render it colorless. * * * * * There is also, to a limited extent, a lessening of the chemical impurity."

On the other hand, although the effect of filtration in rendering water wholesome was not so well known, there was ample precedent for filtering river waters, as it had been practiced in Great Britain since about the year 1811. In 1827 there were enough places where filter-beds were in use in England and Scotland to induce Mr. Thomas Simpson to make a trip of investigation, and he then visited filtration works in various places which were said to have proved completely efficient.

In 1835, a writer, after referring to the entire success of the filter-beds built by Mr. Simpson in 1829, offers this prophecy:

"Probably at no very distant period the desirable practice of filtering the whole of the water supplied from rivers to the inhabitants of great towns for domestic use may be universally adopted."

While this prophecy may have been fulfilled in England from the time when it was made, it is only approaching fulfillment in America.



DARK HOLLOW POND, ADJOINING SPOT POND RESERVOIR.

The first well-equipped plant in America for filtering water through sand was built at Poughkeepsie, N. Y., from plans prepared by the late James P. Kirkwood, Past-President, Am. Soc. C. E., and adopted by the city in 1869. This municipality had the honor of possessing the principal filtration plant of this kind in the United States for many years.

The work of the State Board of Health of Massachusetts, in regard to the purification of water and sewage, is well known, but I should leave out an important part of the development of water supplies and water-supply engineering, were I to omit it; moreover, the organization of this work gives me one of several opportunities to speak from my own experience of the advantages of co-operation between engineers and those engaged along other lines of applied science.

This Board, by a law passed just twenty years ago, was given general supervision of the streams, water supply and sewage disposal in Massachusetts, and was required to advise cities and towns as to the appropriateness of sources of water supply and the best method of disposing of the sewage.

Mr. Hiram F. Mills, the eminent hydraulic engineer, and Chairman of the Committee on Water Supply and Sewerage of the Board, after learning how little definite information was then available on many phases of these subjects, with wonderful foresight initiated the now classical examinations and experiments which have resulted in so great an increase in the knowledge possessed by the water-supply and sewerage engineers of the present day. In this work he was ably supported by the Chairman of the Board, Dr. H. P. Walcott, an eminent sanitarian. The speaker was at the time the Chief Engineer of the Board.

For making these examinations and experiments a staff of chemists, biologists and bacteriologists was engaged, comprising as leaders such men as the late Thomas M. Drown, Professor of Chemistry in the Massachusetts Institute of Technology, and afterward President of Lehigh University, and Professor William T. Sedgwick, of the Massachusetts Institute of Technology; and in the ranks such men as Allen Hazen and George W. Fuller, Members, Am. Soc. C. E.

While this is not an address on sewage purification, I cannot

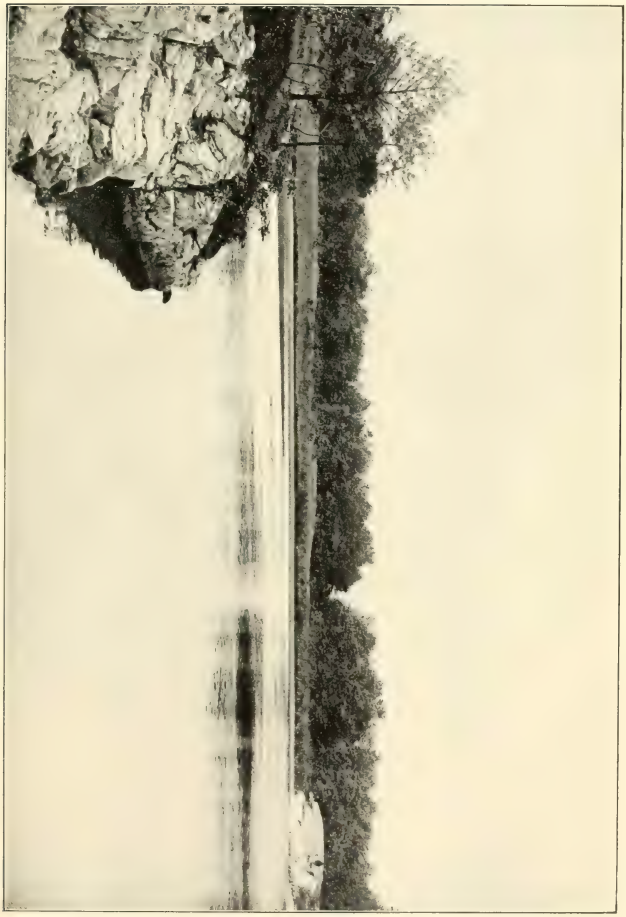
refrain from stating that the experimental work for the purification of sewage and water was planned and executed under the personal direction of Mr. Mills, who, although serving on an unpaid board, gave to this work the larger part of his time for several years.

The examinations of the water supplies of the State, with which the speaker was more intimately associated, and the continued analyses of the waters, proved the great value of the co-operation of engineers, chemists, biologists and bacteriologists, as was recognized by all who participated in the work. These advantages may not be as fully appreciated at the present day, when there is no difficulty in finding individuals who are well acquainted with the engineering, chemistry, biology and sanitary history of water supplies, and who are in many cases experts along more than one of these lines, but men with such knowledge did not exist twenty years ago, and that they are available to-day in this country is to be credited to a large extent to the work of the Massachusetts Board of Health.

Continuing with the subject of the co-operation of engineers with those in other branches of applied science, I wish to express my indebtedness to a skilled geologist, who, from extended examinations at the surface of the ground, with the assistance of only a limited number of borings, predicted with a high degree of accuracy the character of the rock to be encountered in different parts of several aqueduct tunnels; who, in extensive boring operations assisted by his advice as to the best method of obtaining trustworthy samples of the material penetrated, and who, by painstaking work and the resources of the geologist, interpreted the results of some important borings more accurately than they could have been interpreted by the engineers.

It is perhaps unnecessary to refer to the importance of consulting an architect when important buildings are to be erected, because it is now the general rule to do so, but there are more exceptions to the rule than there should be.

It is less common for engineers to consult a landscape architect, as many have faith in their ability to produce pleasing results with geometrical forms. The results of such co-operation in my own experience have been particularly satisfactory, as the



DAM No. 2 AT FELIS RESERVOIR.

carrying out of a landscape architect's plan, made before beginning work, has resulted in pleasing effects at little additional cost, and I will refer at some length to the results of co-operation of this kind in connection with the construction of distributing reservoirs.

Plates XXX to XXXIII explain more clearly than words alone some of the effects produced by the landscape architect's design.

Plate XXX shows a highway along the northeasterly border of a distributing reservoir having an area of 300 acres, which was formed by deepening a natural pond and raising it 9 ft. above its original level. The highway before it was raised had much less pleasing lines than afterward, as it was laid out in nearly straight stretches, and on the water side the bank sloped directly downward from the guard fence at the side of the roadway. In the executed work, the roadway was laid out on easy curves, was kept further from the water, and uniformity of lines and material were avoided, with a view to suggesting natural rather than artificial effects.

Plate XXXI shows the landscape treatment where another highway was necessarily carried across a shallow arm of this reservoir on an embankment. As the portion of the reservoir on the left of the embankment would have been undesirable for water supply, even if deepened, it was left for park purposes, and entirely shut off from the reservoir, which is on the right of the embankment. The advantages of this landscape treatment over a formal treatment with an embankment having a uniform section, steep slopes and guard fences, will be readily appreciated.

Plates XXXII and XXXIII show two of the dams of an artificial reservoir, having an area of 8 acres, located near the summit of elevated land where Nature had formed in part a rock-walled basin requiring only five dams to fill the gaps between these walls. All parts of this reservoir were treated so as to give natural effects. Especial attention is called to Plate XXXIII where, both for landscape effect and to dispose of the large amount of surplus material excavated from the reservoir, a hill as high as any of the surrounding land was created. Buried in this hill is a concrete core-wall extending to a rock foundation, with a rolled embankment of selected earth on both sides of it. In looking at this

reservoir it is somewhat difficult to realize that it is not a natural pond.

Both the reservoirs above mentioned are a part of the Metropolitan Water System of Massachusetts, and are located in a nearly natural woodland park.

The relative advantages of municipal and private ownership of public utilities is a live question in municipal circles at the present time, and in the discussion of this subject water supplies furnish the most valuable object lessons.

In the days of Greece and Rome, the aqueducts and other works were built by the kings and emperors, and operated under their direction. The question of municipal ownership arose in more modern times, when the municipalities became wealthy and to a large extent self-governing.

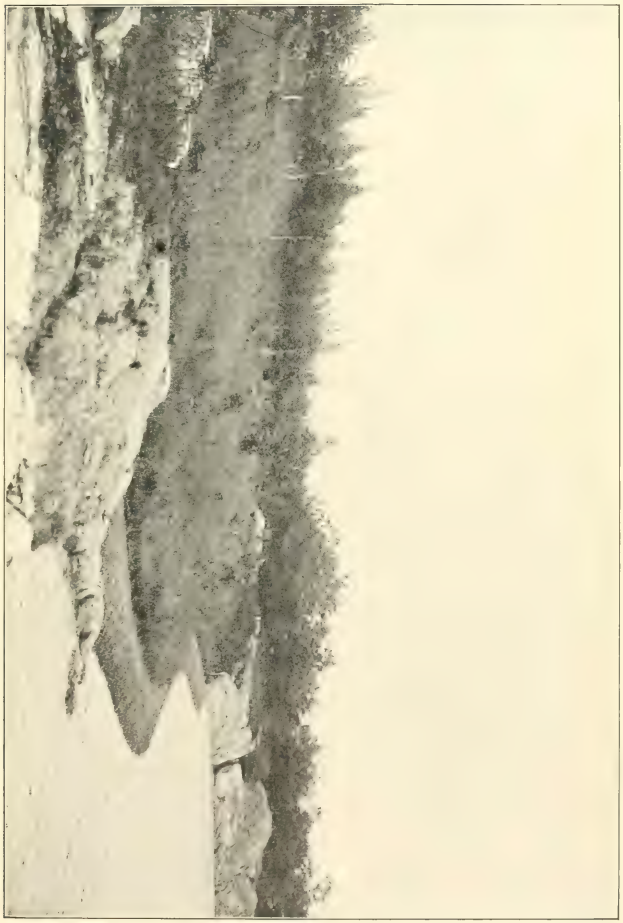
The Hampstead Heath Water Supply, for a portion of London, was built by the municipality and finished in 1590, but about one hundred years after, in 1692, it is recorded that the municipally owned water supplies taken from sources within 5 miles of London were sold to the Hampstead Water Company. After this sale London was supplied by private companies until 1904, when all the immense works for supplying water to that city were again taken by the municipality.

In America the water-works of Philadelphia were municipally owned from the beginning, but the earlier supplies for New York and Boston were introduced by private companies.

There has been for a long period of years a movement toward the municipal ownership of water supplies, and most of the larger water-works in America and in Europe are so owned. The municipally owned water-works, as a rule, have been successful in all respects.

In this address I have endeavored to give a brief account of the development of water supplies and water-supply engineering, covering both water supply as a whole and in some of its more important features.

I have found it necessary to call your attention to the great waste of water as one of the discreditable features of American water supplies, but I have indicated that the responsibility for this lies with the governing bodies of our cities and not with the



DAM NO. 4 AT FELLS RESERVOIR.

water-works engineers. As a result of the great waste of water, however, the engineer has had occasion to build works of unprecedented magnitude, and he has built dams, aqueducts and other water-works structures so well that there have been few failures to record. He has installed pumping engines which are unexcelled in efficiency and capacity, and the works built, as a rule, have been financially successful under municipal as well as under private ownership.

The quality of the water supply of many American cities has been far below the proper standard, but improvements have been effected so rapidly within the past few years that it can be confidently claimed that under the leadership of the engineer and sanitarian a high standard in this respect will soon be reached.

MEMOIRS OF DECEASED MEMBERS.

ANTHONY HOUGHTALING BLAISDELL, M. Am. Soc. C. E.*

DIED SEPTEMBER 9TH, 1905.

Anthony Houghtaling Blaisdell was born at Coeymans, Albany County, New York, on December 23d, 1848. He had some of the best Dutch blood in the country in his veins. On his mother's side he was descended from Colonel Anthony Van Bergen (an officer in the Revolutionary War) and on his father's side from Levi Blaisdell, of Amesbury, Mass., who entered the Revolutionary Army at sixteen years of age as Ensign, and who, at the close of the war, settled in Coeymans, where his descendants still possess a part of the old land grant from Charles I.

Mr. Blaisdell entered Rensselaer Polytechnic Institute, at Troy, New York, at the age of eighteen, and was graduated therefrom with high standing in the class of 1870. In September of that year he entered the service of the United States in the Engineer Department, with which, to a greater or less extent, he was identified during all his subsequent career. His work was largely upon the Mississippi and its tributaries, and, during the greater portion of the time, his headquarters were in St. Louis, Missouri. He rendered important service upon the construction of the Des Moines Rapids Canal, and surveyed and helped carry on improvements on several inland rivers west of the Mississippi. In particular, he was connected for many years with the improvement of the Missouri River and its important tributary, the Osage.

Mr. Blaisdell was an expert boat builder, especially skilled in the ironwork of snagboats and other craft connected with river improvement work, and several of these important boats were built from his designs.

During the greater part of his service with the Government he was the Principal Assistant of Colonel Charles R. Suter, Corps of Engineers, U. S. A. He also served with Colonel Amos Stickney, and to a less extent with other officers of the Corps of Engineers.

In 1879 Mr. Blaisdell went into the private business of boat building in St. Louis, under the firm name of Allen and Blaisdell, and continued in this business until 1885. Owing to various causes, this undertaking did not prove successful, and Mr. Blaisdell returned to the Government service in the latter year. While engaged in private business Mr. Blaisdell was a most active and useful citizen in both educational and philanthropic lines, serving for three years on the St. Louis Board of Education and leaving everywhere

*Memoir prepared by H. M. Chittenden, M. Am. Soc. C. E.

marks of his accurate and organizing mind, no detail being too small to be carefully weighed and its value determined.

In his professional career in the service of the United States Mr. Blaisdell was one of its most trusted employees. He was a man of absolute truthfulness and integrity of character, an earnest and industrious worker, loyal to his superiors, and, all in all, one of those men who make it possible for their employers to accomplish important work. Sincerity marked every act of his life. He was not an ambitious man, in the sense of endeavoring to reach beyond the positions which seemed naturally to fall to his lot, and, whether or not his situation promised him advancement, it made no difference in his fidelity to duty. He was thoroughly beloved by all his employers, and was never lacking a position in the Government services as long as his health permitted him to retain one.

In 1878 Mr. Blaisdell married Miss Mary McConnell, of Chicago, who still survives him. This union was blest with two children, one of whom, Mr. Robert Van Bergen Blaisdell, survives him.

Owing to declining health Mr. Blaisdell left the Government service in 1903 and returned to the parental estate at Coeymans, where he died in 1905, in the house in which he was born.

Mr. Blaisdell was elected a member of the American Society of Civil Engineers on March 3d, 1880.

DAVID MAXSON GREENE, M. Am. Soc. C. E.*

DIED NOVEMBER 9TH, 1905.

David Maxson Greene was born in Brunswick, Rensselaer County, New York, on July 8th, 1832, and was the eldest son of Joseph Langford and Susanna Maxson Greene. His father was a native of Berlin, Rensselaer County, New York, and a descendant of Surgeon John Greene, a purchaser, with Roger Williams, of Indian lands at Providence, Rhode Island. The Rev. John Maxson was a maternal ancestor, born in 1638 at Newport, Rhode Island, the first white child born in the Colony of Rhode Island.

In 1835, his parents moved to Adams, Jefferson County, where he received his early education in the district schools and at Adams Seminary. In 1850 he entered the Rensselaer Polytechnic Institute, where, by great industry, he completed a three-year course in one year, being graduated in 1851 with the degree of Civil Engineer. He was an instructor at the Institute during the following year, and, later, was appointed to a subordinate position in the State Engineering Corps, then engaged in enlarging the Erie Canal. In 1853 he went to Ohio and was engaged on a survey of the Cleveland, Lorain and Wheeling Railroad. The following spring he went to Indiana, where he had charge of a division of the Logansport and Northern Indiana Railroad. He was obliged to return East in the fall on account of ill health. In 1855 he was appointed an Instructor in Railroad Surveying at the Rensselaer Polytechnic Institute, and was shortly afterward elected Professor of Geodesy and Topographical Drawing. In the spring of 1856 he was sent to West Point, and took a special course in topographical engineering under General Thomas H. Neill.

He continued to hold the professorship in Troy until 1861, when he applied for and obtained admission to the United States Navy at $750 a year, although he had a flattering offer of many times that amount from the Peruvian Government. He was attached to the United States Frigate *Susquehanna*, and participated in the attack and capture of the forts at Port Royal and Hatteras Inlet. He was engaged in blockade duty off the Atlantic Coast and in the Gulf of Mexico. In September, 1862, he was detached from the *Susquehanna*, and ordered to the United States Academy at Newport, Rhode Island, as Assistant Professor of Natural and Experimental Philosophy and Instructor in Steam Engineering. In 1865, at his request, he was detailed for duty as Assistant to the Chief of the Bureau of Steam Engineering, in the Navy Department, at Wash-

* Memoir prepared by Franklin A. Hinds, M. Am. Soc. C. E.

ington, D. C. For three years he was engaged in planning and conducting experiments on the physics of steam, and early in 1868 he was appointed a member of the Treasury Commission to examine and test the devices for determining the product of the distilleries of the country. This commission came to be known as the "Whisky Commission." In the spring of the same year he entered the service of the City of Albany, as Chief Assistant Engineer in charge of surveys for the extension of the water-works. In February, 1869, he was detailed as First Assistant Engineer in charge of the United States Steamship *Narragansett,* for a cruise in the West Indies, but yellow fever broke out in May, and the ship was sent back into northern waters and detained in quarantine at the Isle of Shoals, near Portsmouth, New Hampshire, for two weeks. In September, he was detailed as First Assistant Engineer in charge of the *Frolic,* the Port Admiral's vessel, in New York Harbor. He reported for duty, and at the same time tendered his resignation from the naval service.

He then returned to Troy and engaged in the general practice of his profession. Early in 1870 he was employed to examine the Ottawa River, and report to the Canadian Parliament on the probable effect of the sawdust and mill refuse discharged by lumbermen at Ottawa. In 1871 he examined plans for a water supply for the City of Troy, and advocated the pumping system, which, after a controversy extending over several years, was finally adopted. About this time he was elected Engineer of the New York State Commission to test devices to substitute steam for animal power on the canals. In January, 1874, he was appointed Engineer in charge of the Eastern Division of the State Canals. In July of the same year he was appointed Deputy State Engineer, in which office he continued until 1877, when he again returned to Troy and resumed general engineering practice. In August, 1878, he was elected Director of the Rensselaer Polytechnic Institute, a position which he held for thirteen years, resigning in 1891. During that period he was engaged in a general engineering practice extending through eleven different States, the District of Columbia and Canada. In 1879 and 1880, he was Chief Engineer of the Troy water-works extension. He had a wide experience as an expert witness on hydraulic and steam engineering in the Courts. He was a School Commissioner of Troy from 1873 to 1876.

He was the author of the following books: "Notes on Steam Engineering" (1886); "Method of Laying out Easement Curves on Railroads" (1891); "The Fly Wheel as a Regulator" (1890); "Manuscript Notes on Map Projections;" "Practical Hydraulics" and "Railroad Notes for the Use of Students."

It is interesting to note that many of the Japanese students in

engineering, who were graduated from the Rensselaer Polytechnic Institute and who have since risen to places of importance in their own country, remember with gratitude their association with Professor Greene, and have personally expressed their appreciation when on return visits to this country.

Professor Greene was a Member of the American Society of Naval Engineers, the New England Society of Naval Engineers, the Society of Naval Architects and Marine Engineers, the American Association for the Advancement of Science, the International Association of Navigation, a Fellow of the National Geographical Society, a Member of the Society of the Founders and Patriots of the United States, Sons of the American Revolution, Military Order of the Loyal Legion of the United States, Naval Order of the United States, Post John A. Griswold, G. A. R., the Army and Navy Club of New York City, and the Ionic Club of Troy; he was also a Mason, of the 32d degree. He was a member of General Joseph B. Carr's staff, with the rank of Colonel.

He was married at Adams, New York, January 31st, 1855, to Maria N. Skinner, a daughter of the late Hon. Calvin Skinner, of Adams, who still survives him. His home in Troy was at 41 First Street, but he died at his summer home, in Adams, New York, on November 9th, 1905.

Professor Greene was essentially an engineer. He lived in an engineering atmosphere, his mind always drifted toward scientific subjects, and he kept well abreast of the age in which he lived. Impatient of carelessness and unfaithfulness, his name became a synonym for accuracy and reliability, and he was a strict disciplinarian. He possessed a strong character, was a scientific worker, and, among his associates, a genial gentleman.

David Maxson Greene was elected a Member of the American Society of Civil Engineers on May 20th, 1868.

GABRIEL LEVERICH, M. Am. Soc. C. E.*

DIED NOVEMBER 28TH, 1905.

Gabriel Leverich was born on a farm about five miles from Elmira, New York, on August 19th, 1834. He was the son of Samuel and Sarah Leverich, for many years residents of New York City.

In his early years he attended what was then called the District (public) school in the vicinity, where he excelled in all his studies.

His taste seemed to be decidedly for mechanical work, and among his early inventions were a hay-rake, much like the one now generally used; a hay-fork for use by horse power, and other ingenious and practical inventions.

Two intimate friends and near neighbors, of about his own age, were graduates of the Rensselaer Polytechnic Institute, and this led him also to attend the institute, from which he was graduated in 1857.

His first engagement, after graduation, was at the Trenton Locomotive Works, where machinery for the manufacture of small arms was about to be introduced. In this connection he spent some time at the works near Springfield, Massachusetts, obtaining data from which he designed and constructed the necessary tools.

At this time Mr. Wiard was at Trenton, engaged in the construction of heavy ordnance embodying proposed improvements, and Mr. Leverich became interested in the matter. This, at a later date, led to his employment at Boston in the design of the "Thompson gun." The writer does not remember the details of this, but Mr. Leverich was one of the first to shrink on a jacket or insert a lining. This involved the refined accuracy of measurement with which we are now so familiar.

Other enterprises with which he was intimately connected were: the design and construction of apparatus for the destructive distillation of wood, in which all the products were saved; also, of machinery for the manufacture of fuel briquettes, by the compression of peat; and of improvements in the propulsion of tram cars.

Mr. Leverich was elected a Member of the American Society of Civil Engineers on July 6th, 1870, and from 1872 to 1877, inclusive, served as Secretary of the Society. In these trying years of the Society's history, while nominally receiving a fair salary, he spent a considerable proportion of it in paying expenses which he deemed essential for its advancement, and for which the income was insufficient. Only his most intimate friends knew of this, however.

*Memoir prepared by Francis Collingwood, M. Am. Soc. C. E.

His services upon the first East River Bridge began when the approaches were under construction. He was responsible for the general features of the design of the Franklin Square Bridge, where the complexities of construction were very considerable. He took an important part, also, in the design of the New York station; and had entire charge of the changes, in both the New York and Brooklyn stations, made necessary by the great increase in traffic. These have more than doubled the carrying capacity of the bridge.

The rapid growth of traffic made it necessary also to improve the apparatus for propelling the cars on the bridge and increase its power. In carrying this out, Mr. Leverich showed an accurate knowledge of mechanical devices, and of the principles of mechanics; and the machinery is a model of ingenuity and effectiveness. This apparatus is described in a paper by him in Vol. XVIII of the *Transactions* of the Society. He contributed several other papers and discussions to the *Transactions*. All are noted for their clearness and accuracy of thought and expression. This was characteristic of all his work, and enabled him to answer all objections and carry his plans through to completion.

He died at the age of 71, having been an invalid for about six years, leaving a widow and a married daughter.

Mr. Leverich was bright and cheerful in disposition, and a good conversationalist. He was generous to a fault, and often, to his own detriment, helped others. He kept fully posted in all professional, industrial and political matters.

In early life he joined the Episcopal Church, and was always interested in its welfare.

JAMES MacNAUGHTON, M. Am. Soc. C. E.*

DIED DECEMBER 29TH, 1905.

James MacNaughton was born on January 6th, 1851, in Albany, New York, and died on December 29th, 1905, in New York City. He was the son of Dr. James MacNaughton, Dean of the Faculty of the Albany Medical College.

His early education was received at the Albany Boys' Academy, Albany, New York, from which institution he was graduated in 1867. After a year's study, he entered the sophomore class of Yale College, and was graduated with honors from the classical department in July, 1871. While there he paid particular attention to the study of mathematics, and had conferred upon him the second senior mathematical prize of his class.

After graduation he accompanied Professor Marsh as a member of his party on a geological exploration in Kansas, Colorado and Wyoming. On his return to Albany from this expedition he studied chemistry and other scientific branches at the Albany Medical College. In 1873 he entered the Rensselaer Polytechnic Institute, and took a special course in technical engineering subjects for a period of two years.

In 1875, immediately after leaving the Rensselaer Polytechnic Institute, he was appointed Rodman on Mr. C. L. McAlpine's party on surveys for the new aqueduct for New York City. He was engaged in the field, and also on the office work in connection with this survey, and remained with the party until the completion of the maps, estimates, etc., in the fall of the next year.

In April, 1876, he was appointed Rodman in the Department of Public Works, New York City, in connection with the construction of the new storage reservoir near Brewster, New York. Soon after his appointment to this position, he was made Leveler on the same work, and was thus engaged for about a year and a half.

In the autumn of 1877, he was promoted to be an Assistant in charge of the surveys for the location of a new storage reservoir, east of Brewster Station, but in the latter part of December, 1877, he resigned and returned to Albany, where he took charge of superintending and getting out the plans for the Hotel Kenmore, in Albany. He had charge of this work as Superintending Engineer of construction, and in January, 1879, as soon as the work was completed, he went abroad, and for four months was engaged in study in the École des Ponts et Chaussées, at Paris.

He returned to the United States in October, 1879, and shortly

* Memoir prepared by James C. McGuire, Assoc. M. Am. Soc. C. E.

thereafter was engaged as Assistant Engineer on the West Shore Railroad for about two years.

In 1885 he was appointed an engineer on an expedition sent out by the Canadian Government on H. M. S. *Alert,* which made explorations and surveys on the Hudson Bay Coast.

Mr. MacNaughton was a member of the Association for the Preservation of the Adirondacks, and took much interest in the development of the forests and the cutting of timber from large tracts of land. In 1903 he took a course at the Yale Forestry School.

He was elected a Member of the American Institute of Mining Engineers in 1890. The late President Blanco of Venezuela decorated him for services in that country, in connection with certain engineering enterprises. He was a Member of the New York Board of Trade and Transportation of New York City.

Just prior to his death, he did much to develop the manufacture of Ferro-Titanium on a commercial basis, having been President of the Ferro-Titanium Company which built a plant at Niagara Falls and successfully manufactured Ferro-Titanium for the market. He received much recognition for his work in this line, both in the United States and abroad, and it is entirely due to his efforts that the use of this alloy has been made possible from a commercial standpoint. He was also President of the MacIntyre Iron Company at the time of his death.

Mr. MacNaughton was a member of the Arts Club, the University Club, and the Down Town Association, of New York City; the Tahawas Club, of Essex County, New York, the Fort Orange Club, and the Albany Country Club, of Albany, New York.

It is to be particularly noted that both in his business connections, and in the societies and clubs of which he was a member, he was always conspicuous for his dignified bearing and courteous treatment of all who came in contact with him, and especially those under him. He never married.

While his loss to his friends is great, his loss to the scientific world is even greater, for he was engaged in the development of properties and industries of which his knowledge was so complete, and for which he had done so much that it will be impossible to fill his place. His death is mourned by all who knew him or who came in contact with him, either as a friend or as a citizen.

Mr. MacNaughton was elected a Member of the American Society of Civil Engineers on May 5th, 1880.

WILLIAM MARSHALL REES, M. Am. Soc. C. E.*

DIED DECEMBER 4TH, 1905.

William Marshall Rees was born at Stroudsburg, Pennsylvania, on December 24th, 1851, and died at Memphis, Tennessee, on December 4th, 1905.

He was graduated from Lehigh University in 1874 at the head of his class. Almost immediately after graduation Mr. Rees went with the East Sugar Loaf Colliery, of Stockton, Pennsylvania, as Assistant Superintendent and Engineer, and remained with them in that capacity until July, 1875, when he left to accept the position of Superintendent of the Humboldt Colliery at Hazleton, Pennsylvania.

In January, 1877, he left Hazleton, to engage in railroad and mining work. During 1877-78 he located and constructed the Stroudsburg and Bethlehem Railroad. During the same time he was Mining Engineer for G. B. Linderman and General Manager of the Bethlehem Iron Company.

In December, 1878, Mr. Rees left his native State to go south and engage in Government work on the Mississippi River under the Mississippi River Commission, being attracted by the high scientific character of the work.

During the first three years of his Government service, he was engaged on surveys, examinations, gauging the river and gathering other data on which to base plans for its improvement. During 1881 he was engaged in constructing snagboats to be operated on Red River. In 1882 he went with the Pratt Coal and Coke Company, of Birmingham, Alabama, as Superintendent. In 1883 he returned to the Government service on the Mississippi River and, as Principal Assistant Engineer, had charge of all channel work in the First and Second Districts. During his Government service he designed and constructed various floating plants used in connection with channel improvement.

In the latter part of 1889 Mr. Rees went with the Sanitary District of Chicago as Assistant Chief Engineer. Upon the resignation of the Chief Engineer, L. E. Cooley, M. Am. Soc. C. E., Mr. Rees also resigned. He then returned to his old position in the Government service, where he remained until his death, the result of injuries received in the performance of duty.

During his different engagements Mr. Rees did a good deal of expert work, being eagerly sought by those desiring such advice, which he was eminently fitted to give. He was a man of broad education, and possessed a wonderful fund of information in all

* Memoir prepared by W. M. Gardner, M. Am. Soc. C. E.

branches of the profession, which he was ever ready to impart to the younger members.

It has been the writer's good fortune to be associated with Mr. Rees for the past ten years, and he feels himself indebted to him for many kindnesses received from the helping hand so generously extended to smooth over rough places.

Mr. Rees was a Charter Member and Past-President of the Memphis Engineering Society. He was elected a Member of the American Society of Civil Engineers on October 4th, 1905.

ARCHER COCHRAN STITES, M. Am. Soc. C. E.*

DIED AUGUST 27TH, 1905.

Archer Cochran Stites was born at Middleton, Delaware, on April 22d, 1863, and resided there until the autumn of 1883, when he entered the class of '87 at the Rensselaer Polytechnic Institute, of Troy, New York. He was graduated with his class in June, 1887, and went at once to Kansas City, Missouri, to enter the employ of Mr. John W. Nier, hydraulic and mechanical engineer.

After spending a few months in Mr. Nier's office, he entered that of J. A. L. Waddell, M. Am. Soc. C. E., who was then representing The Phœnix Bridge Company and the Phœnix Iron Company in that portion of the United States lying west of the Mississippi River, and at the same time was establishing a practice as Consulting Bridge Engineer. Mr. Stites remained in this position for more than four years, or until the summer of 1892, when Mr. Waddell resigned both agencies and suggested Mr. Stites as his successor, his suggestion being accepted. Mr. Stites' first duty in this new position was to settle the unfinished business relating to the completion of the Elevated Railroad in St. Louis, which The Phœnix Bridge Company had built for the St. Louis Terminal Railroad Association. In the autumn of 1892 he settled in Chicago, where for nine years he represented very successfully The Phœnix Bridge Company and The Phœnix Iron Company.

At the time he took this new position, the modern office building, with its heavy metal framework, was just beginning to be developed, and a number of important structures of this type were soon built. There were also then several large bridge projects under consideration. Mr. Stites secured, for the companies he represented, a number of these important structures, and, by his conduct of the business involved, fully established the standing of these companies in Chicago and the West.

In 1901 he resigned these agencies in order to enter the employ of Joseph T. Ryerson and Son, and to inaugurate for them an engineering department with special reference to structural steel work. For two years he labored unceasingly on this work; but in September, 1903, his health failed, and he was forced to go to Asheville, North Carolina, to recuperate. From Asheville he went to various other places in the South, reaching Covington, Louisiana, in December, 1904.

The plans evolved by Mr. Stites for Joseph T. Ryerson and Son, which were on an unusually large scale, have been successfully carried out since he left their employ.

* Memoir prepared by J. A. L. Waddell, John Sterling Deans, Members, Am. Soc. C. E., and J. V. Norcross, Esq.

His health improved so materially at Covington that he arranged to re-enter business, with headquarters at New Orleans, Louisiana, but he was stricken suddenly with malarial fever, and on August 27th, 1905, died at Monteagle, Tennessee.

Mr. Stites' acquaintance throughout the Middle West was unusually large, for he made lasting friendships easily. His genial, kindly nature, overflowing with good fellowship, demanded contact with his fellows at many points, and make him universally popular. This characteristic, so attractive to his many friends, was enhanced in his home, where he was a most loving husband and father.

He represented, to an eminent degree, a type of engineer that has been generally recognized only of late years, *viz.*, the Commercial Engineer. While his education and early practice were entirely professional, he soon evinced a decided preference for the business side of engineering, and, during the latter half of the four or five years that he spent with Mr. Waddell, a large portion of the business of the office fell to his lot. Since 1892 his experience had been mainly in the line of commercial engineering, and in it he was eminently successful. Had his health held out, and had he lived to old age, he undoubtedly would have become a financier of recognized standing.

In 1892 Mr. Stites married Miss Louise Sells, daughter of the late Mr. Luke Sells, of St. Louis. Three children were born of the marriage, all of whom are living.

Mr. Stites was elected a Junior of the American Society of Civil Engineers on December 3d, 1890, an Associate Member on November 4th, 1891, and a Member on May 6th, 1896. He was also a Member of the Technical Club of Chicago, and at one time served as its Secretary.

JOHN WALKER BARRIGER, Jr., Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 19TH, 1902.

John Walker Barriger, Jr., was born in Washington, D. C., on July 20th, 1874. He was the third son of General John Walker Barriger, of the United States Army, and Sarah Frances Wright Barriger, and was descended from a distinguished revolutionary ancestry.

His early education was conducted in the schools established at the various posts where his father was stationed. Later, he attended the High School of St. Louis, and Washington University.

In the early autumn of 1894, when the Terminal Railroad Association, of St. Louis, was building the new Union Station, involving elevated railroad construction and bridge work, he was employed as Assistant to the Resident Engineer on this work, as well as on field work and maintenance of way.

In December, 1895, he was with a party on the preliminary and location surveys for the Kansas City, Pittsburg and Gulf Railroad, through Louisiana, as rodman, office man and compiler of estimates.

On the completion of this survey he returned to St. Louis and resumed the study of engineering, in order to equip himself more thoroughly for professional work. During this period he also undertook land line work in the vicinity of St. Louis.

On the completion of his studies, in April, 1897, Mr. Barriger returned to work in the field, and was Assistant to the Engineer in charge of the Texas Division of the St. Louis Southwestern Railway, with headquarters at Texarkana, Texas.

In May, 1898, he became Assistant Engineer, and was put in responsible charge of maintenance of way, with an office in Tyler, Texas. This position he held until December, 1899. He then went to Kansas City as Engineer in the office of E. Holbrook, M. Am. Soc. C. E., Chief Engineer of the Kansas City, Pittsburg and Gulf Railroad (later the Kansas City Southern Railway), and had charge of the estimates for track, bridge and building work, also masonry construction.

In this office he had many and various duties, as many improvements in the road were being made.

In March, 1902, he resigned this position to accept one of greater responsibility, entering the employ of the St. Louis, Memphis and Southeastern Railroad as Bridge Engineer. His duties were the supervision of the checking of stresses in the bridges to be erected, and the examination and approval of bridge plans. He also had charge of erection.

* Memoir prepared from papers on file at the House of the Society.

Mr. Barriger's professional associates esteemed him as a man, and recognized, in his steady advance to positions of increasing responsibility, the thoroughness and quality of his work. He was an accurate and skilful draftsman, and this talent was by no means confined to engineering, for he had a keen appreciation of art.

His personal character was unselfish, and he was always kindly and considerate for others.

He met his death at the hands of a man whom he had befriended.

In 1899 he was married to Miss Edith Beck, of St. Louis, who, with one son, survives him.

Mr. Barriger was elected a Junior of the American Society of Civil Engineers on May 31st, 1898, and an Associate Member on April 2d, 1902.

VAN DUSEN HITE-SMITH, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 27TH, 1905.

Van Dusen Hite-Smith was born on August 4th, 1874, at Louisville, Kentucky, and belonged to one of the oldest families of New Albany and Madison, Indiana.

When he was fourteen years of age his father failed in business, and it became necessary for him to strike out for himself, and this he did, with the courage of a man of mature years.

His first work was as a chainman on surveys for the Union Railroad, of Chattanooga, on which he remained for three months. From October, 1888, to April, 1893, he was engaged with Messrs. Guild and White, Engineers, of Chattanooga, Tennessee, in successive grades from Rodman to Resident Engineer. From May to December, 1893, he was engaged with the United States Engineer Corps on the improvements of the Tennessee River. He then returned to Messrs. Guild and White, and was engaged as Assistant Engineer on surveys of the Lookout Mountain Incline, the Ætna Coal Mines, and on railroad and municipal work. In 1894 he superintended the construction of water-works at Cynthiana and at Morganfield, Kentucky.

Late in 1893 he purchased a limestone rock quarry from which he shipped fluxing material to the Citico Furnace at Chattanooga.

Realizing his need of a thorough education, Mr. Hite-Smith entered the University of Tennessee in 1895, and in 1897 was graduated with honors, being not only the first man in his class, but the only man who ever took the engineering course in that university in two years.

After graduation he was employed by Messrs. Guild and White in the construction of water-works at Chester and Union, South Carolina.

In July, 1898, he was engaged by the Standard Boiler and Bridge Company, of Bellaire, Ohio, in the construction of water-works at Sharpsville, Pennsylvania; and in September, 1898, he took a contract for the construction of water-works and a sewerage plant at Pocomoke City, Maryland. Following this he was connected with the construction of water-works at St. Petersburg and West Tampa, Florida.

In April, 1900, he formed a partnership with Mr. Frederic Minshall, under the firm name Hite-Smith and Minshall, Civil and Sanitary Engineers, and up to the time of his death was engaged as Consulting Engineer or in the construction of a number of water-works and sewerage systems in the South.

* Memoir prepared from papers on file at the House of the Society.

Mr. Hite-Smith accomplished more than is often done in so short a life. He was a man of great physical strength and mental vigor and did not spare himself in his attempts to surmount every obstacle. The great tax which he placed upon his body and mind is thought to have been the cause of his breakdown and death, which occurred at his home in Knoxville on August 27th, 1905.

Mr. Hite-Smith was elected an Associate Member of the American Society of Civil Engineers on April 30th, 1901.

GEORGE DRAPER STRATTON, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 21ST, 1905.

George Draper Stratton was born in Orange, New Jersey, on June 5th, 1870. When he was one year old his father moved with his family to Newburgh, New York, where he was interested in the Washington Iron Foundry. On the death of his father, in 1876, his mother moved to Riverside, California, with her children.

At the age of 15 he was forced to leave school in order to help support his mother and sisters. This he continued to do until his mother's death, in 1887.

Mr. Stratton used the small share which he received from his mother's estate in obtaining a college education. He entered Stanford University in the fall of 1891, with the pioneer class, as a special student. By hard work he made up his entrance deficiencies, and was graduated with honor, in the course in Civil Engineering, in May, 1895.

He at once commenced the practice of his profession, and, after several minor engagements, entered the service of the Southern Pacific Railroad Company. He was Assistant Engineer on the Sacramento Division for one year, and was then made Roadmaster at Marysville. Here his health became undermined by malaria, and at the end of the year he was transferred to the Western Division of the Southern Pacific Railroad, with headquarters at Oakland, California. He remained there for six years, or until death called him, on November 21st, 1905. His record as Assistant Engineer and as Assistant Resident Engineer was excellent.

Mr. Stratton was married, on January 17th, 1899, to Miss Jeannie Gift. His wife and a daughter, four years of age, survive him.

Mr. Stratton was a devoted churchman. Two years before his death he became a vestryman of St. Andrew's Church, Oakland. By his beautiful example while occupying that office, he was a source of great help to many. Mr. Stratton was a Mason and a Knight Templar. Among his associates, both business and social, he was much beloved for his fine character and sweet temper. He was ever and always the same quiet, even-tempered man, whose sincerity and loyalty were unfailing. The profession has lost in him a good engineer; the world, a good man.

Mr. Stratton was elected an Associate Member of the American Society of Civil Engineers on October 4th, 1899.

* Memoir prepared by Charles D. Marx, E. M. Boggs, Members, Am. Soc. C. E., and R. M. Drake, Assoc. M. Am. Soc. C. E., from information furnished by Mrs. Stratton.

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